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SPILLWAY TESTS CONFIRM MODEL-PROTOTYPE CONFORMANCE

By A. J. Peterka

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SPILLWAY TESTS CONFIRM MODEL-PROTOTYPE CONFORMANCE

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INTRODUCTION

There is a general need for data which can be used to compare the performance of models and prototypes and extend the range of usefulness of hydraulic models as an aid in design. Ordinarily, prototype data are difficult to obtain and usually the model data are not in the prototype range of heads or discharges, which makes direct comparison difficult. Also, several years may elapse between model tests and the time the prototype is subjected to large flows. The Heart Butte and Shadehill Spillways, however, operated during the first flood season following their completions and almost immediately after the hydraulic model tests were made. With the test data on the models still fresh it was possible to obtain prototype data on short notice that could be compared with model tests.

This monograph compares the performances of both the Heart Butte and Shadehill Dam morning glory spillway models with the performances of the prototype structures. The results of these comparisons add further proof to the premise that prototype performance can be predicted with accuracy from model tests.

Brief discussions are given of the necessary hydraulic model tests conducted on the scale models to aid in the design of the structures and to obtain data useful in operating the prototype structures. Following a description of the 1950 flood on the Heart River and the 1952 flood on the Grand River, which produced discharges of 68 percent and 88 percent, respectively, of the maximum anticipated flows, the performances of the prototype structures are described.

Direct model-prototype comparisons are made of spillway performance and discharge for free and submerged conditions; spillway air demand; stilling basin performance, including erosion downstream from the basin; and tail-water elevations in the excavated channel. Photographs and charts are used to illustrate the agreement found between model and prototype performance.

Certain aspects of the prototype performances which are beyond the scope of model tests are also discussed. This includes the effect of ice completely covering the morning-glory during submerged discharge, the erosion of the downstream riverbanks, and the effectiveness of the riprap used on the excavated channel banks. The results of inspections of the spillway tunnels and structures following the floods are also given.

HEART BUTTE DAM STUDIES

Description of Project

Heart Butte Dam is on the Heart River 60 miles west of Bismarck, North Dakota, and is part of the Heart River Unit of the Missouri River Basin Project (figure 1). The dam is of compacted earth fill with a rock riprap cover, rises 135 feet above stream bed, and is 1,860 feet long. It serves both irrigation and flood-control purposes. At maximum water-surface elevation the reservoir will contain 392,500 acre-feet of water collected from a drainage area of 1,810 square miles.

Figure 2 shows plan and sections of the flood-control spillway and of the outlet works, which, as the transparent model in figure 3 clearly shows, is an integral part of the spillway structure. The spillway is a morning-glory type, 32 feet 6 inches in outside diameter. It discharges into an 11-foot diameter vertical shaft and a 90° diverging elbow that lead to a nearly horizontal tunnel 14 feet in diameter and about 800 feet long. Around the vertical shaft of the spillway is the entrance to the outlet works, which discharges into a 5-foot 3-inch diameter tunnel located directly above the spillway tunnel and from there into the larger tunnel from above. From the junction point the spillway tunnel carries both discharges into the hydraulic-jump stilling basin.

The morning-glory spillway is unusual in that it is designed to operate throughout the range of free discharge, the transition range between free and submerged discharge, and up to submergences as great as 53.7 feet of water above the crest.* The spillway crest is equipped with six equally spaced piers placed radially in plan, but does not have control gates of any kind. The outlet works discharge is controlled at the lower end of the

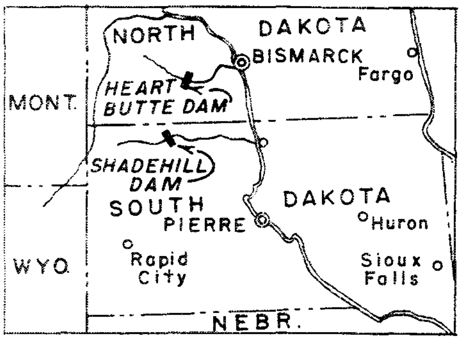
*To the author's knowledge, Heart Butte and Shadehill spillways are unique in this respect.

outlet works tunnel by a 4- by 5-foot high-pressure slide gate.

The capacity of the spillway is 5,450 second-feet at maximum reservoir elevation 2118.2. The maximum vertical fall from headwater to stilling basin floor is about 130 feet. The capacity of the outlet works is 650 second-feet at spillway crest level elevation 2064.50.

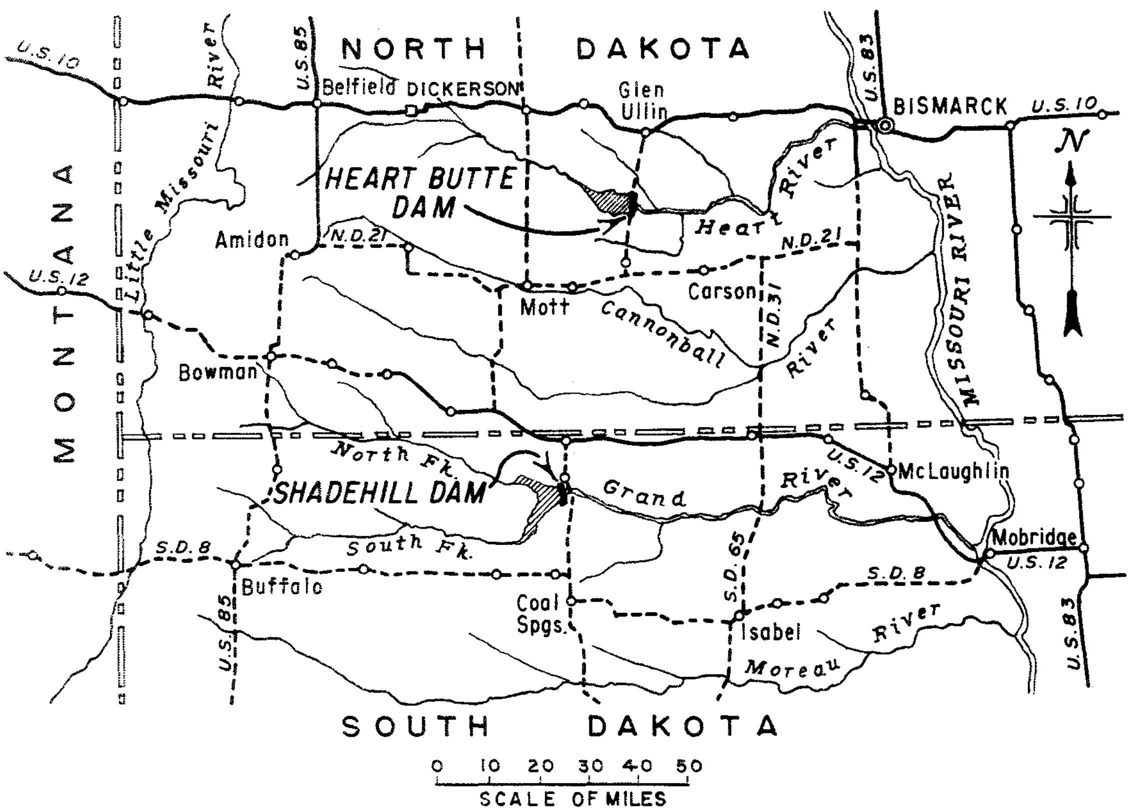
Model Tests

The discharge structures were tested on a 1:21.5 scale model. The morning-glory, the outlet works intake structure, and the surrounding topography were constructed



INDEX MAP

FIGURE 1--Map of portions of North Dakota and South Dakota that shows the locations of both Heart Butte and Shadehill Dams.



VICINITY MAP

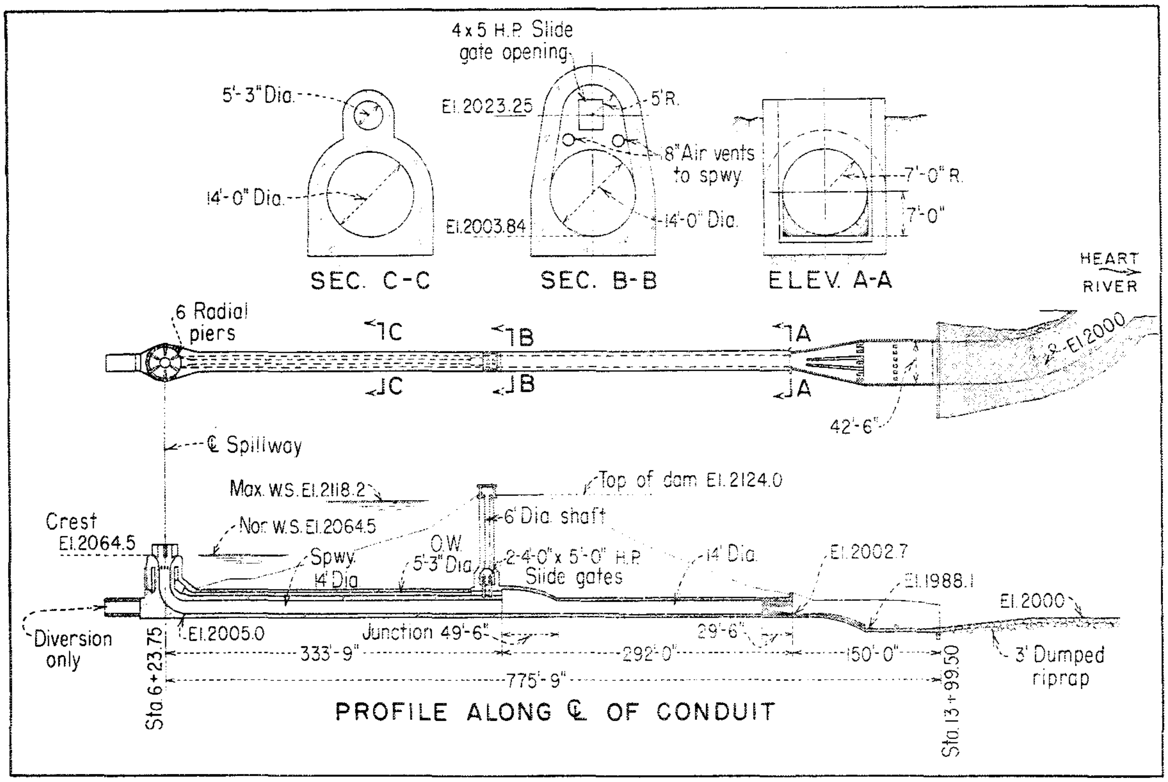
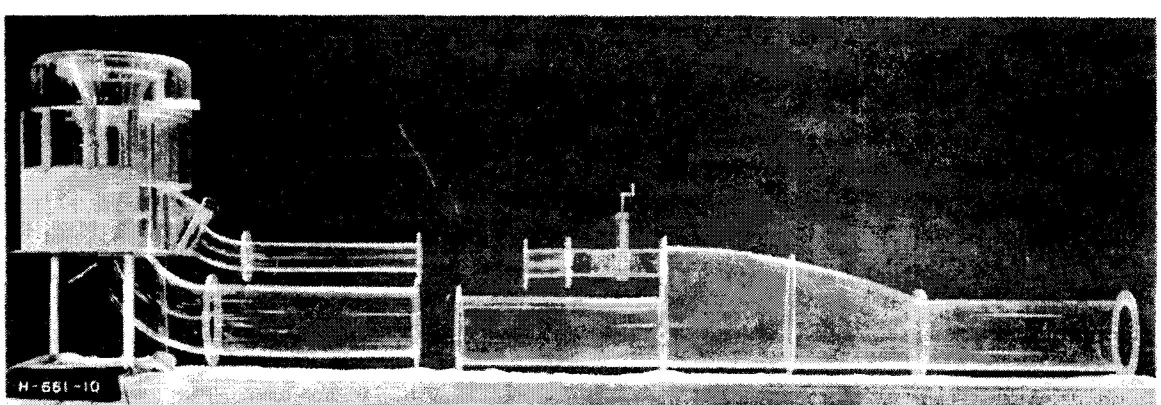


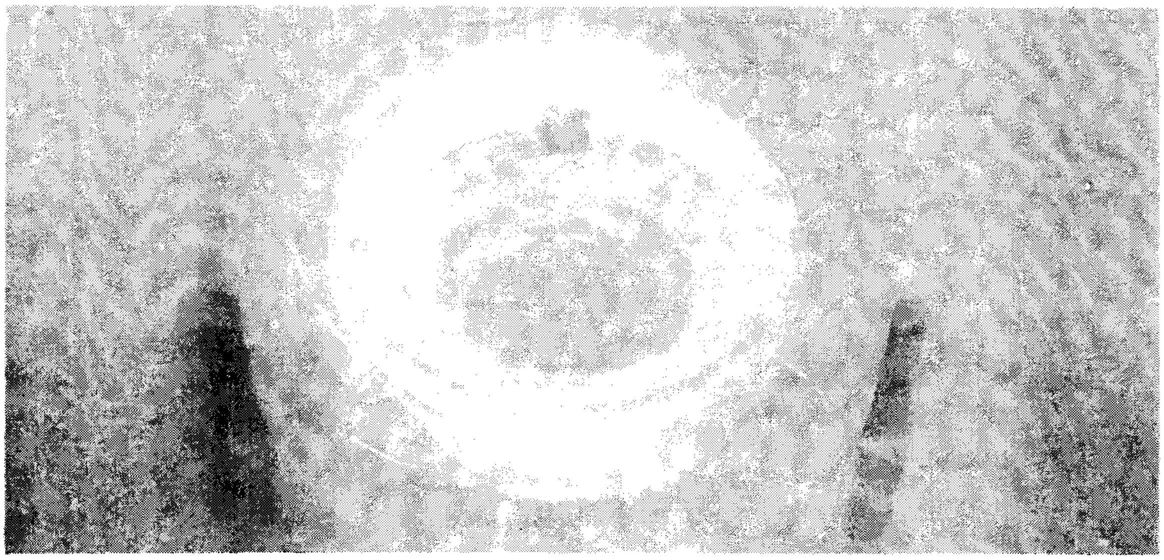
FIGURE 2--Plan and sections of the spillway and outlet works at Heart Butte Dam.

within the head box. The two tunnels, including the outlet works control gate, the 90° vertical bend, and the tunnel junction section, were built outside the head box. The stilling basin and a portion of the down river topography were constructed within the tail box. Much of the structure was modeled in transparent plastic to permit direct observation of flow conditions.

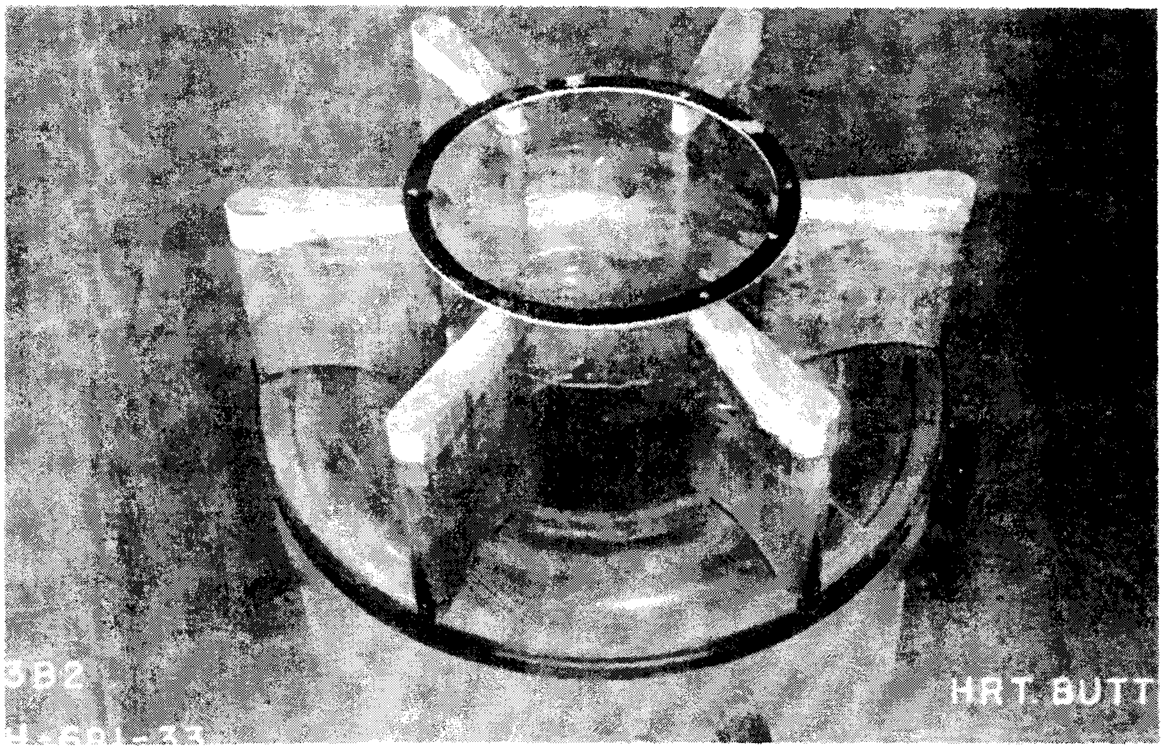
1. Spillway and pier tests.--Tests on a preliminary design of the morning-glory spillway indicated that the discharge capacity of the structures was larger than necessary. Con-

FIGURE 3--The essential parts of the laboratory model of the spillway and outlet works at Heart Butte Dam were molded of transparent plastic. The intake for the outlet works surrounds the vertical spillway shaft and the discharge is controlled by the slide gate. Model scale was 1:21.5.





A--A violent vortex formed in the spillway at heads exceeding the submergence point. The tail of the vortex extended downward into the horizontal tunnel.



B--Six piers placed radially on the crest reduced the vortex to negligible size.

FIGURE 4--Tests of the morning-glory spillway model for Heart Butte Dam at a discharge of 3,750 cfs.

sequently, the vertical shaft diameter was reduced from 14 to 11 feet. This required the reshaping of the spillway profile to fit the vertical shaft, and the installation of a 90° vertical transition bend at the bottom of the shaft. The discharge capacity was then found to be approximately correct, according to the irrigation and flood control requirements.

Vortices which formed in the model when the spillway was submerged, figure 4A, were thoroughly investigated both experimentally and mathematically. Since these same vortices could form to scale in the prototype, attempts were made to eliminate them. Various arrangements of piers, dividing walls, and floating and fixed rafts were tested. As a result, six spillway crest piers were recommended for use on the prototype, see figure 4B. It was found unnecessary to extend the piers as high as the maximum headwater elevation, a distance of 54 feet. Since vortex action diminished rapidly when the head on the crest approached 14 feet, it was necessary to extend the piers only to this height.

2. Deflector and vertical bend tests.--Tests to determine the most satisfactory type of vertical bend showed that a diverging elbow joining the 11-foot-diameter shaft with the 14-foot-diameter horizontal tunnel had a distinct advantage because it provided greater space between the water surface and the tunnel crown for ventilating the vertical bend from the atmosphere at the tunnel outlet.

However, even with this arrangement, difficulty was encountered in preventing the horizontal tunnel from filling unexpectedly when the spillway and outlet works were both operating. Flow passing through the bend did not break cleanly from the crown of the bend. The flow had a tendency to follow the crown throughout the bend, causing a change in the location of the flow control. When the control moved downstream the head on the system increased, causing an increase in discharge which filled the tunnel. This, in turn, caused a still greater head with a correspondingly greater discharge and resulted in negative pressures of considerable magnitudes on the spillway face. Once the tunnel had filled, it was impossible to obtain open channel flow again unless the head on the spillway was reduced to a point considerably below where it had filled. To correct this condition a small deflector was placed at the base of the vertical shaft on the downstream, or crown, side of the shaft, see figure 5. The deflector accomplished three things: (1) it provided a positive control at the base of the vertical shaft and prevented the tunnel from filling, (2) it had a stabilizing effect on smaller flows and provided a flat water surface on all flows passing into the vertical bend, and (3) it provided a clear passage for air to circulate as far upstream as the base of the deflector. The thickness of the deflector at the base was varied in the model to determine the size necessary to exactly meet the discharge requirements at certain heads since precise tests had shown that the 11-foot-diameter vertical shaft was slightly too large. Spillway flow in the tunnel was found to be satisfactory after the structure had been modified as described. Figure 6 shows the flow entering, passing through, and leaving the vertical bend with the deflector in place. Note the smooth and flat water surface on the flow entering the tunnel.

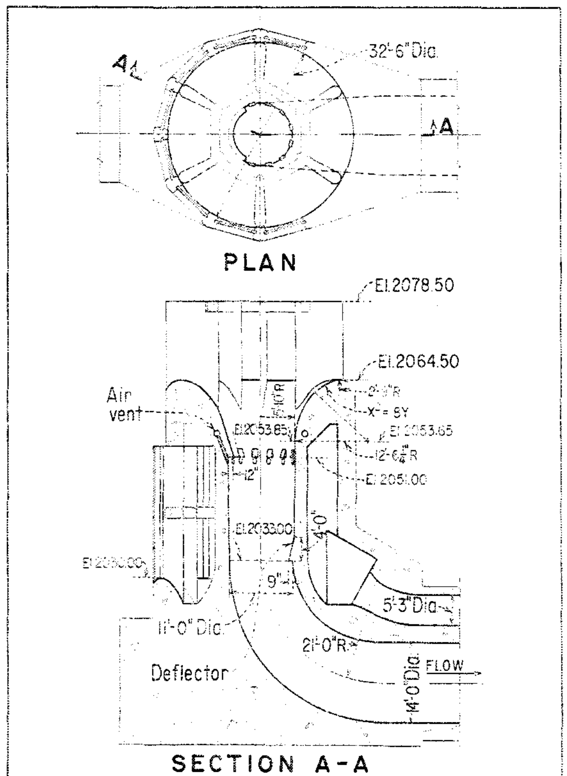


FIGURE 5—Entrance details of the spillway and outlet works at Heart Butte Dam.

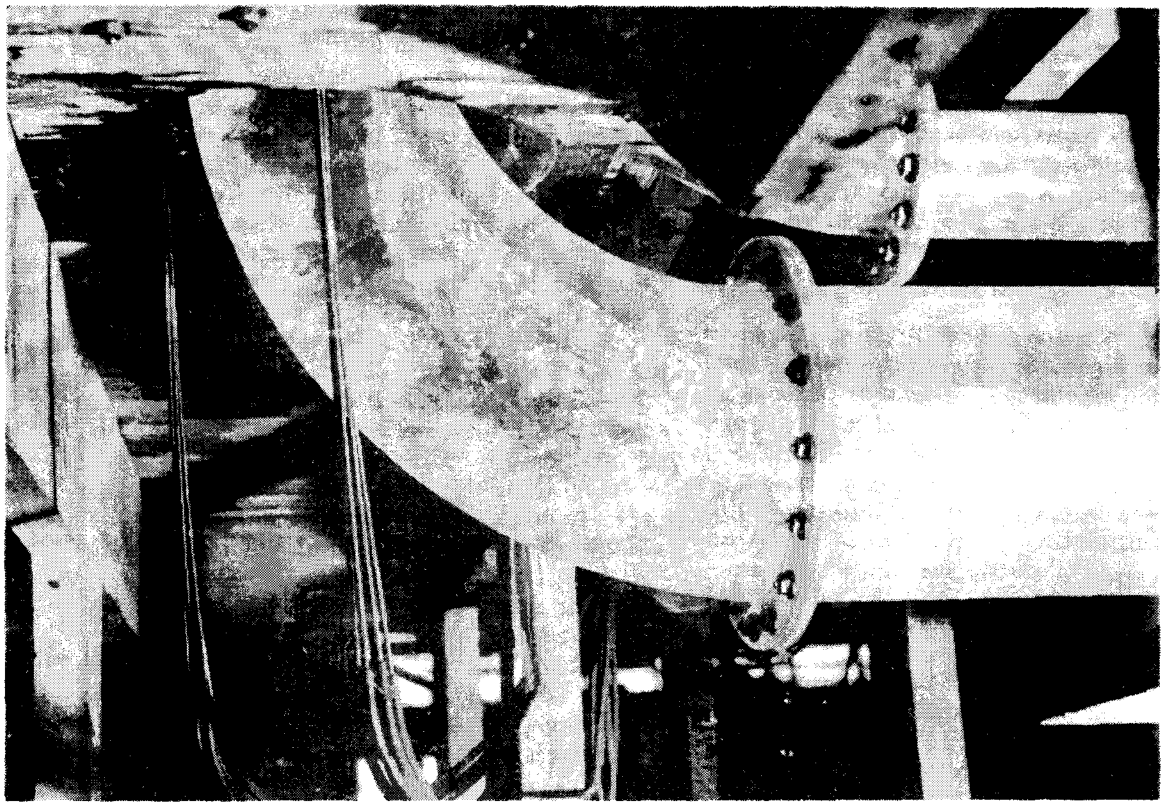
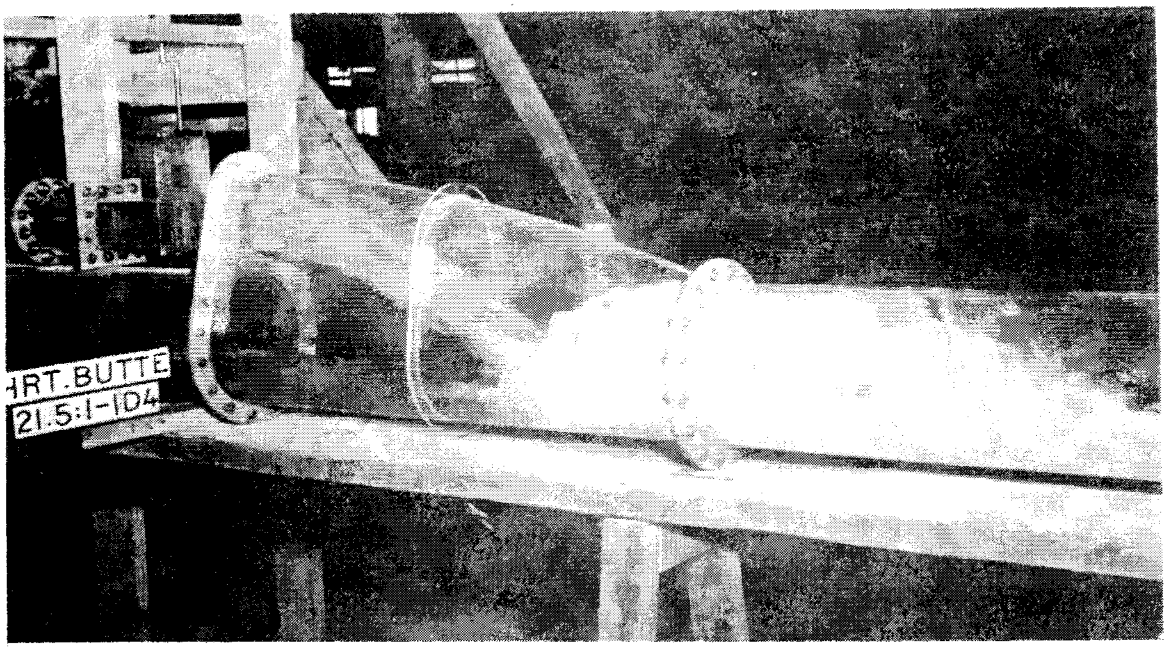


FIGURE 6--The deflector at the base of the vertical shaft produced smooth flow and a flat water surface in this test of the vertical bend at 3,750 cfs. Although the air flow appeared continuous to the eye, high-speed photographs showed that air entered the flow in bursts.

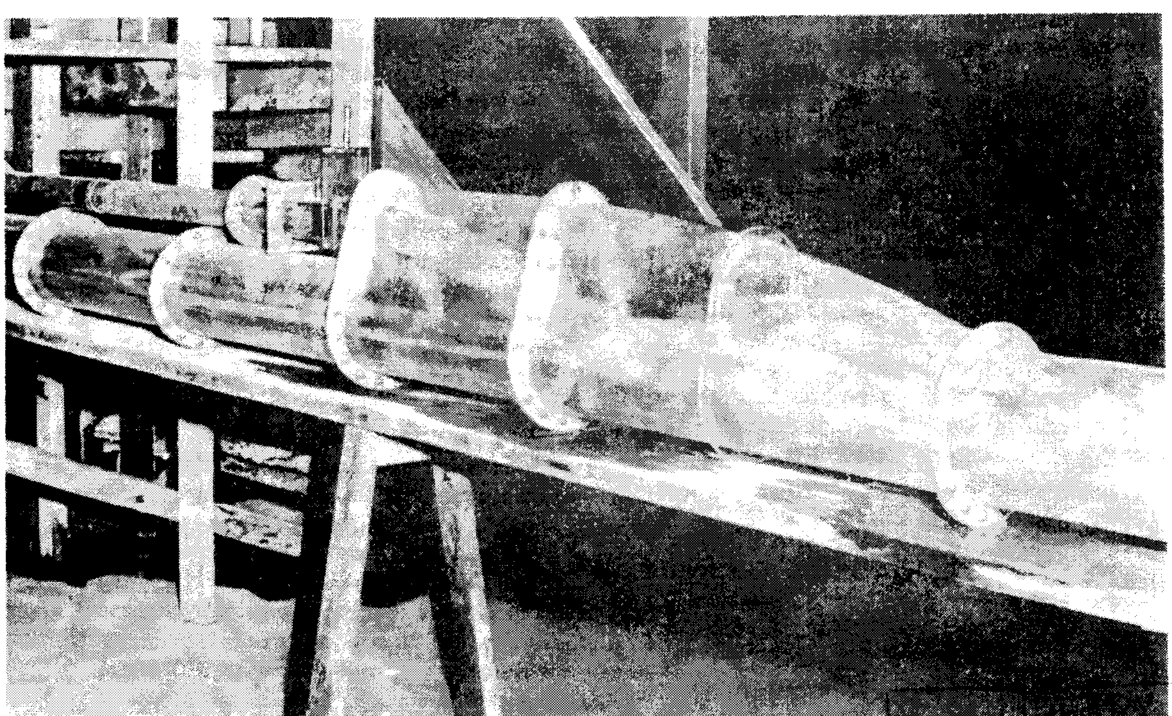
3. Outlet works and tunnel junction tests.--The outlet works discharge entered the spillway tunnel from above in a junction section as shown in figure 7A. The jet in striking the tunnel bottom caused a "piling up" of water as shown in the photograph. This caused no difficulty unless the spillway was operating at maximum or near maximum flow. In the high spillway discharge range, the resistance resulting from the pile-up in the junction section caused the spillway tunnel to fill, which was undesirable.

A longer transition was tested in an attempt to eliminate the rough water at the entrance to the main tunnel, and some improvement was obtained, see figure 7B. With the longer transition the tunnel did not fill quite so readily. Other benefits were minor in nature, however, and it was decided to use the shorter transition with the operating instructions that the outlet works gate be closed when the spillway was discharging. The larger transition did not provide sufficient improvement to warrant its extra cost.

4. Stilling basin tests.--An effective energy-dissipating device was required in the stilling basin because of the friable nature of the material in the river channel and riverbanks. Even moderate erosion tendencies and wave heights could not be tolerated. Consequently, it was felt that a hydraulic jump basin would be necessary to provide good energy dissipation and a smooth water surface in the downstream channel. The first stilling basin tests indicated that the main problem was concerned with spreading the high-velocity water, about 60 feet per second, into a uniformly distributed sheet suitable for the formation of a jump. The first attempt to induce lateral spreading was by means of a sudden rise in the stilling basin floor downstream from the tunnel portal. It was found that a hump sufficiently long to produce even a moderate amount of spreading resulted in an extremely long stilling basin structure. With a basin of reasonable length, sufficient spreading could not be produced to permit the formation of an effective jump. The problem was solved by discharging the flow onto a



A--The short junction section recommended for use in the prototype caused some disturbance in the flow at the entrance to the spillway tunnel.



B--A considerably longer junction section was tested, but its greater cost was not believed to be justified, even though smoother flow resulted.

FIGURE 7--Tests of the junction of the outlet works and spillway tunnels for Heart Butte Dam at an outlet works discharge of 850 cfs.

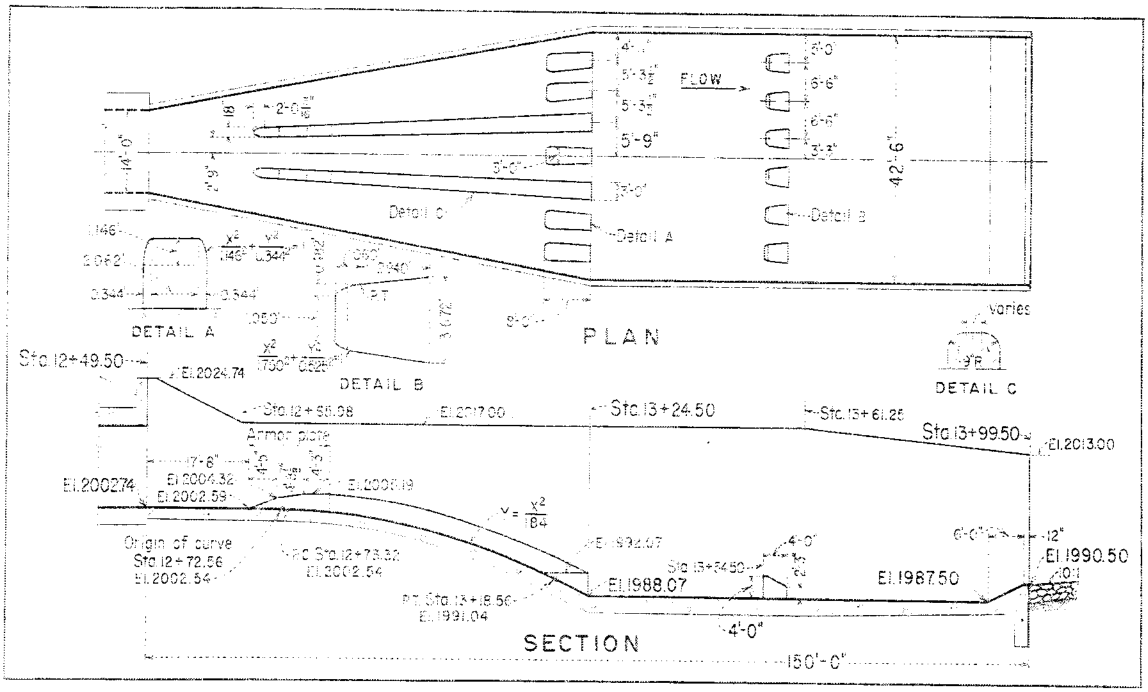


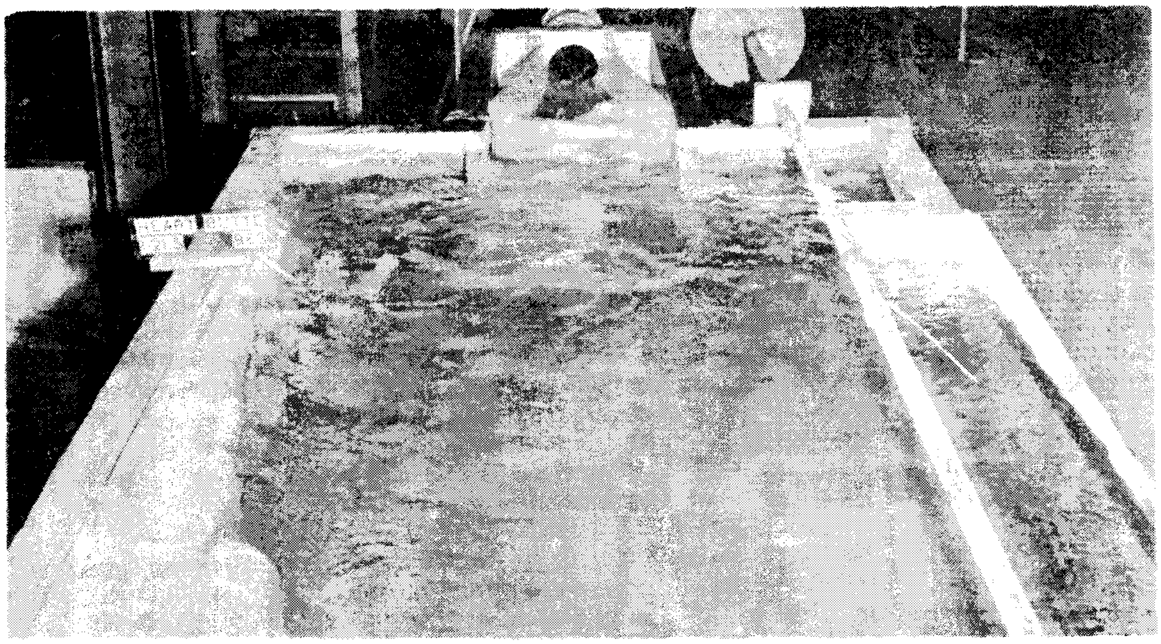
FIGURE 8--Stilling basin details for the spillway and outlet works at Heart Butte Dam.

horizontal floor about 23 feet long after it had passed through a transition section at the end of the tunnel which started the spreading of the flow, see figure 8. The flat floor then induced more spreading before the flow dropped downward on the trajectory curve. Tests showed that this arrangement produced good lateral distribution of flow as far downstream as the trajectory curve and fairly good distribution beyond this point. The addition of two low walls, placed so as to divide the basin approximately into thirds, produced excellent downstream distribution of flow and an efficient hydraulic jump in the basin. The walls, which varied from 3 to 4 feet high throughout their length, did not extend upward through the flow for high discharges but produced the desired effect of distributing the flow from 14 feet wide at the tunnel portal to an ultimate 42.5 feet wide in a horizontal distance of 75 feet.

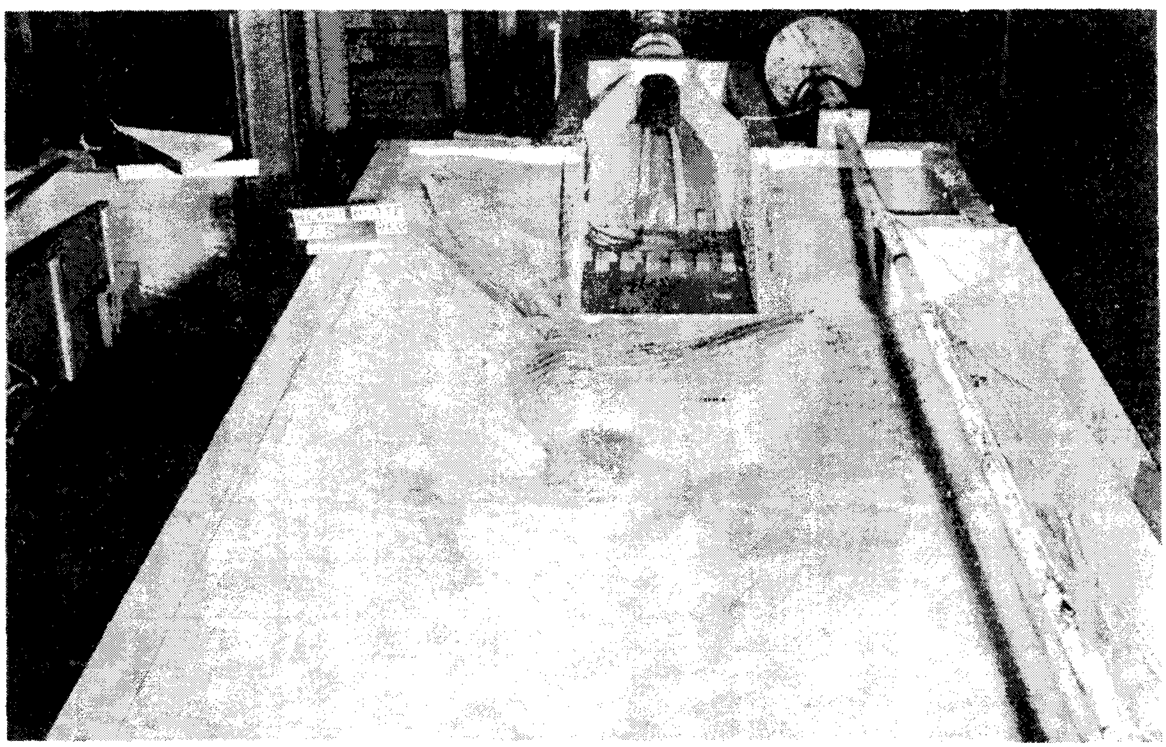
Chute blocks and baffle piers were used to increase the fine grain turbulence in the basin and thereby reduce the required length of the stilling basin. The shape of the baffle piers, dividing wall noses, and trajectory curves were modified to provide atmospheric pressures or above on critical areas, since tests on preliminary designs had indicated that pressures as low as 18 feet of water below atmospheric pressure occurred downstream from sharp corners. The recommended stilling basin is shown in figure 8.

The performance of the developed stilling basin was evaluated from erosion tests made on a movable bed located downstream from the model basin and from wave height observations made in the excavated tailrace channel. Erosion tests were made using a well-graded sand (100 percent passed a No. 4 sieve and 3 percent passed a No. 50 sieve). These tests showed that erosion tendencies were less severe on the channel bottom than on the sloping sides. Wave action originating in the hydraulic jump combined with a slight surging action caused rapid decay of the banks. Every effort was made to keep the waves and surges to a minimum, but it was deemed necessary to riprap the banks of the prototype. Figure 9 shows the performance of the recommended basin.

5. <u>Spillway air tests</u>.--When the morning-glory spillway was designed, it was an-



A--The recommended basin, shown in operation, was reduced in dimensions to the minimum consistent with acceptable performance.



B--Although some erosion occured, the operation was considered acceptable, since riprap protection was to be used in the prototype. The model produced the effect shown in a half-hour's operation.

FIGURE 9--Tests of the model stilling basin for Heart Butte Dam at a discharge of 5,600 cfs and at tail water elevation 2012.

ticipated that air introduced into the spillway discharge at a point just below the spillway crest might help to cushion the impact of the flow passing around the vertical bend. It was important that unnecessary impact and vibrations caused by the flowing water be eliminated, because the entire structure was to be constructed on sand. Furthermore, if for any reason cavitation should occur in or near the vertical bend, the presence of the entrained air might reduce the tendency to damage the concrete tunnel lining. Laboratory tests have shown that even very small quantities of air introduced into the flow will delay the appearance of cavitation damage.*

Model tests on the many devices proposed to increase the entrained air in the flow showed that only a relatively small amount of air entered the flow regardless of how the air-entraining devices were arranged. However, it was known that air flow in small hydraulic models is uncertain and that a greater percentage of air can be expected to enter a similar prototype structure. The amount of increase to be expected in the prototype is not known and could not be computed since the factors governing the entrainment of air are not known. After tests on many different model arrangements, it was finally decided to construct the prototype air vents shown in figure 5 and to provide measuring facilities in the prototype structure so that air quantity determinations could be made. Figure 6 shows the vertical bend discharging 3,750 cfs with air introduced by the air deflectors entrained in the flow. To the unaided eye the air flow appeared continuous but in the 1/15,000-second exposure photograph the air is seen to enter in gusts. This was more clearly illustrated in the extremely slow motion pictures (3,000 frames per second) made of this condition.

*A. J. Peterka, The Effect of Entrained Air on Cavitation Pitting, Proceedings, Minnesota International Hydraulics Convention, 1953, University of Minnesota, Minneapolis, Minn.

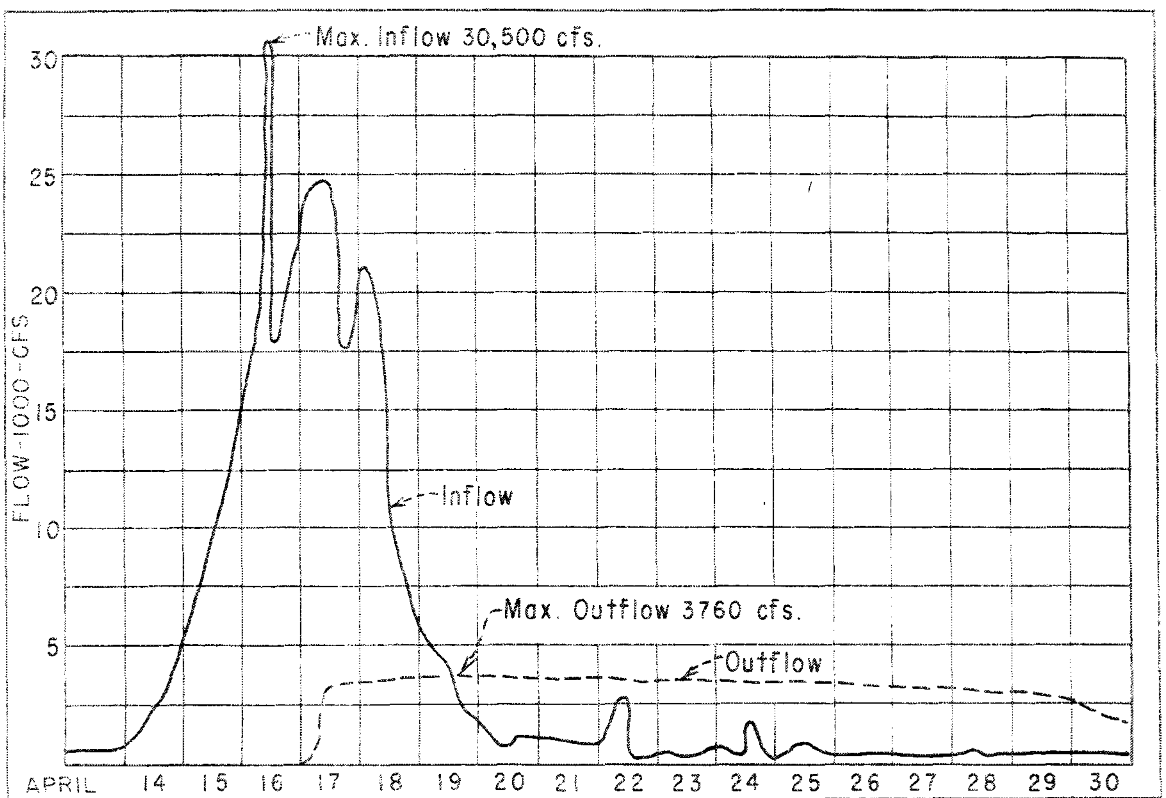


FIGURE 10--Hydrographs of the flood on the Heart River at Heart Butte Dam in April 1950.

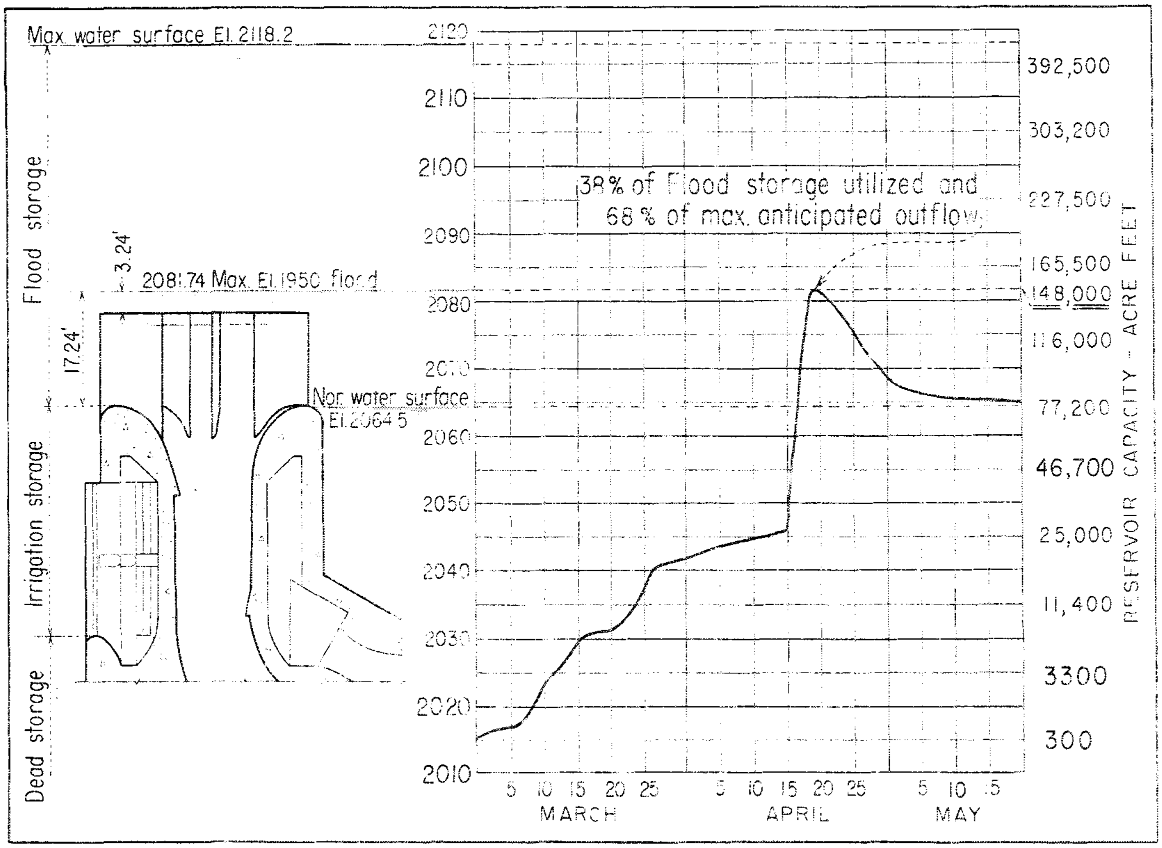


FIGURE 11--Reservoir elevations and hydraulic data of the 1950 flood at Heart Butte Dam.

The 1950 Spring Flood

Preceding the heavy run-off in April 1950, the weather had been cold and the ground was frozen and covered with snow. A stiff wind had blown the snow off the ridges, concentrating it on the slopes and in the valleys of the drainage area. The weather then turned unseasonably warm, causing a fast melt and heavy run-off from the frozen terrain. On April 15, 1950, the temperature reached 80°, and the snow melt caused an increase in the inflow to the reservoir from 5,000 to 31,500 cfs on April 16, see figure 10. The high run-off and inflow continued throughout April 17 and most of April 18. The spillway went into operation on April 17, reaching a peak flow of 3,760 second feet on April 19 and continued without appreciable reduction in discharge through April 29, a period of over 12 days. The maximum outflow discharge represented 68 percent of the anticipated maximum outflow, and the maximum reservoir elevation indicated that 38 percent of the flood storage had been utilized. Figure 11 shows the hydraulic data in terms of the spillway elevations. At the time of maximum outflow, the spillway crest was submerged 17.24 feet, making the reservoir elevation 3.24 feet over the tops of the spillway piers, see figure 11. The maximum height of fall, headwater to tail water, was 72 feet, and the energy entering the stilling basin amounted to 21,000 horsepower.

The Heart River, on which Heart Butte Dam is constructed, flows into the Missouri at Mandan, North Dakota, about 6 miles from Bismarck, figure 1. Some flood damage occurred at Mandan, caused primarily by high water in the Missouri River. Both rail and highway travel were impossible during the high water. The Heart Butte Dam undoubtedly reduced the flood crest in Mandan, but no figures are available as to extent. The structure operated as intended and therefore provided as much protection as was anticipated.

Model-Prototype Comparison Tests

It was recognized that model-prototype comparison data pertaining to the spillway discharge and the air demand would be particularly valuable and that comparisons of the erosion in the excavated channel, wave heights below the stilling basin, and profiles below the stilling basin would also be of interest. In the course of recording these data, other comparisons were made which included observations on vortex formation above the spillway and a comparison of the computed and actual tail-water curves in the excavated channel and in the river. Water-surface profiles in the stilling basin and data on the riprap protection were also obtained.

1. Spillway capacity.--During the 1950 run-off, readings were taken each morning and afternoon on the headwater gage located in the gate operating house. These are shown plotted in figure 11. Using the discharge-capacity curve obtained from the model tests on the morning-glory spillway, figure 12, an outflow hydrograph was prepared, see figure 13. On April 17, 19, 25, and May 1 the United States Geological Survey made stream gage measurements in the river downstream from the stilling basin to determine the discharge of the spillway. During these measurements, the irrigation outlet works was closed. The

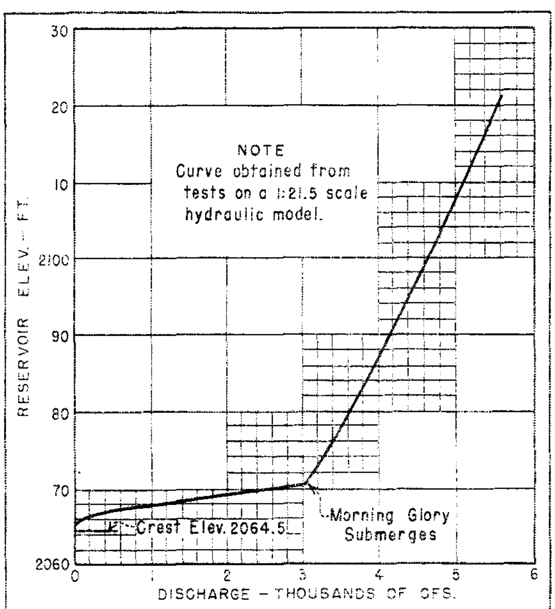


FIGURE 12--Spillway discharge capacity curve for Heart Butte Dam.

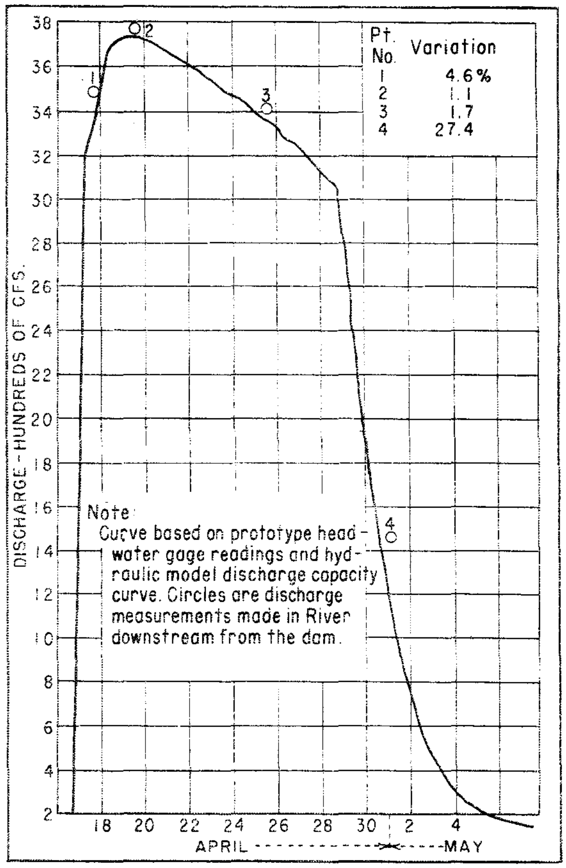
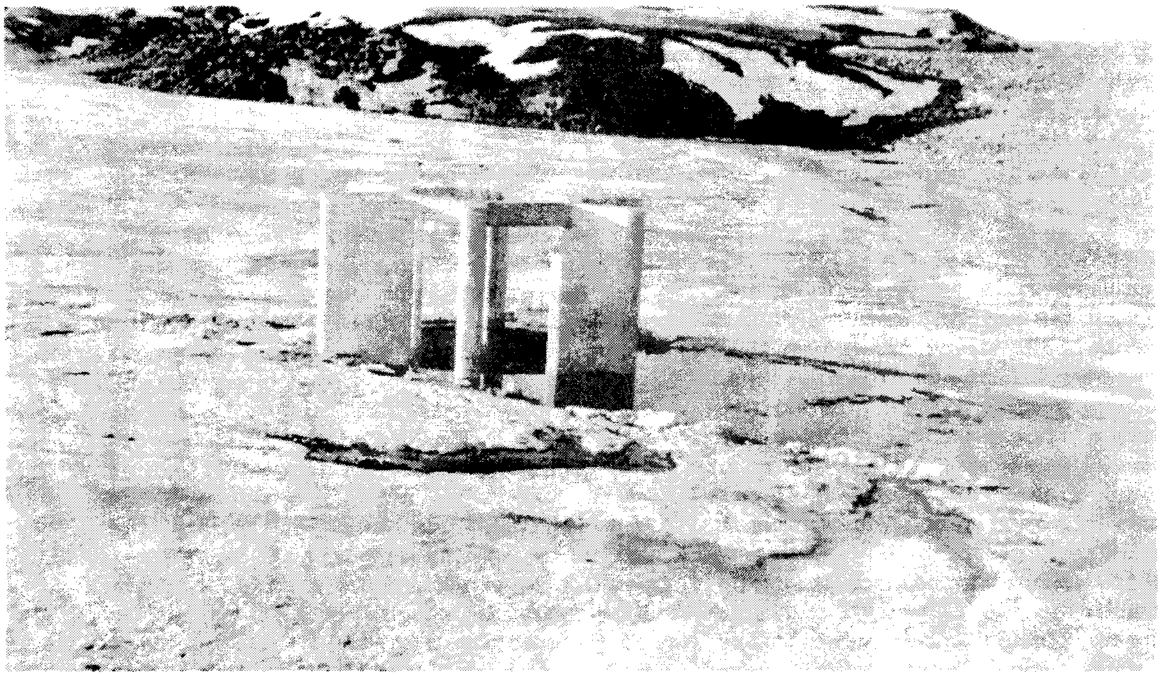
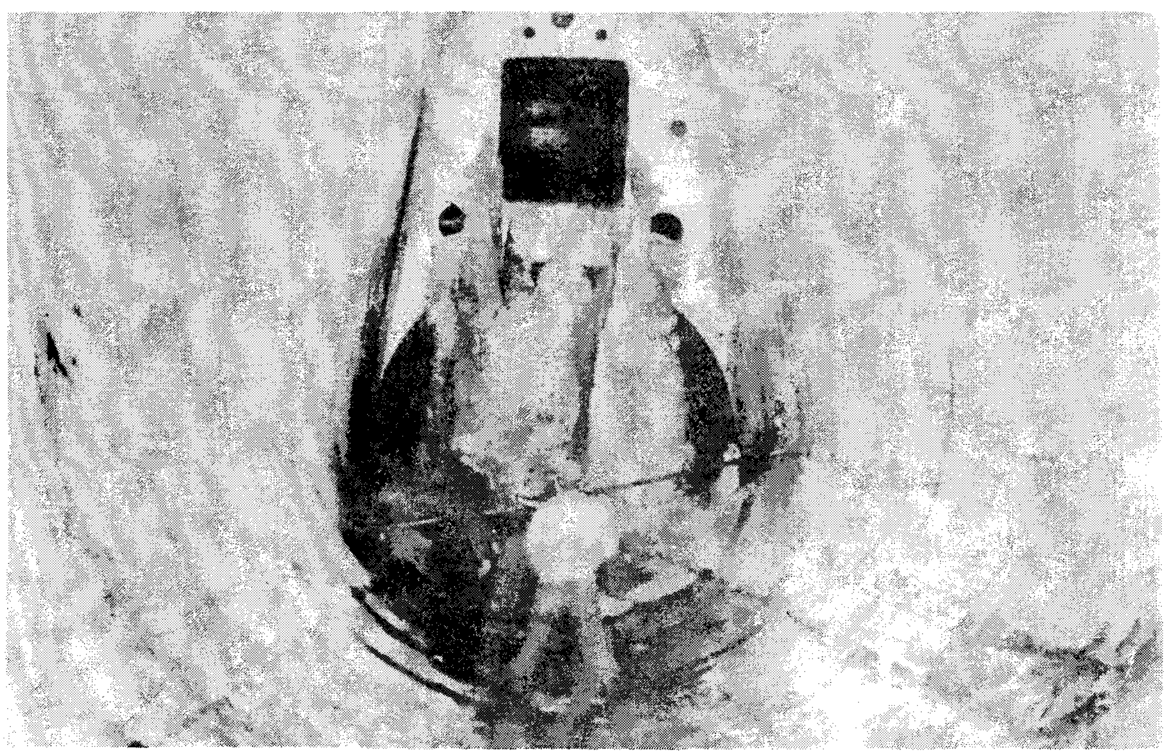


FIGURE 13--Discharge comparison of Heart Butte model and prototype.

discharges determined by the United States Geological Survey, indicated by circles on figure 13, indicate the degree of agreement between the model and the prototype measurements. Differences were 4.6, 1.1, and 1.7 percent for the April 17, 19, and 25 determinations, respectively. For all practical purposes, these points indicate good agreement between model and prototype discharge characteristics. On May 1 the difference was 27.4 percent, indicating considerable disagreement. The measurements on April 17 and May 1 were not made under ideal conditions. The United States Geological Survey notes for April 17 indicated that ice in the channel may have affected the measurements, and on May 1, when the greatest disagreement was found, that a wind was blowing which might have altered the relation between the head on the crest and the headwater gage reading. Another possible cause for the discrepancy might be the rapidly falling stage in the reservoir during the measurements on May 1 as indicated in the hydro-



A--On April 16, 1950, the reservoir had risen to the spillway crest, elevation 2064.5. The upstream face of the dam is at the right.



B--Ice that had formed from leaks around the outlet works control gate was removed before the spillway operated.

FIGURE 14--Performance of spillway and outlet works at Heart Butte Dam.

graph of figure 13. In general, however, the agreement between model and prototype discharges is considered excellent, particularly at the higher discharges, and it is believed that the rating curve obtained from the model will adequately serve to determine discharge values through the prototype morning-glory spillway.

2. Spillway performance--Free and submerged discharges.--During the model tests it was noticed that, for certain arrangements of the structure, the transition from free to submerged flow and vice versa was accompanied by violent surging in the vertical shaft. In some cases the unstable flow condition existed over several feet of change in reservoir elevation. A mushroom-shaped column of water rose and fell in the shaft, causing excessive splashing and turbulence. In addition to giving poor hydraulic conditions it was feared that the prototype structure would be subjected to objectionable forces and vibration. Consequently, the structure recommended for field construction was developed by tests to provide a minimum transition range, i.e., less than 0.2 foot. The rating curve determined by model tests, see figure 12, indicates the definite change from one type of flow to the other. It was for this reason that the prototype spillway was closely observed when the headwater reached the transition range.

On April 16, 1950, the reservoir had risen to the spillway crest elevation 2064.5. Ice covered most of the reservoir area, but there was some open water close to the spillway, see figure 14A. Before flow started over the spillway, the tunnel was inspected and ice which had formed around the outlet works gate was removed, see figure 14B. By April 17, 1950, the reservoir had risen sufficiently to submerge the spillway and provide a head of 9.3 feet on the crest, corresponding to a discharge of 3,250 cfs, see figure 15. Sometime during the night the reservoir elevation had passed through the critical region where the flow changes from free to submerged. Some ice had been discharged through the spillway, but it had caused no apparent difficulty. On April 18, the piers were completely covered and the reservoir was covered with ice, see figure 16A, which appeared to be about 12 inches thick. A small amount of trash had collected over the spillway and slight movement of the trash was the only evidence that the spillway existed. The

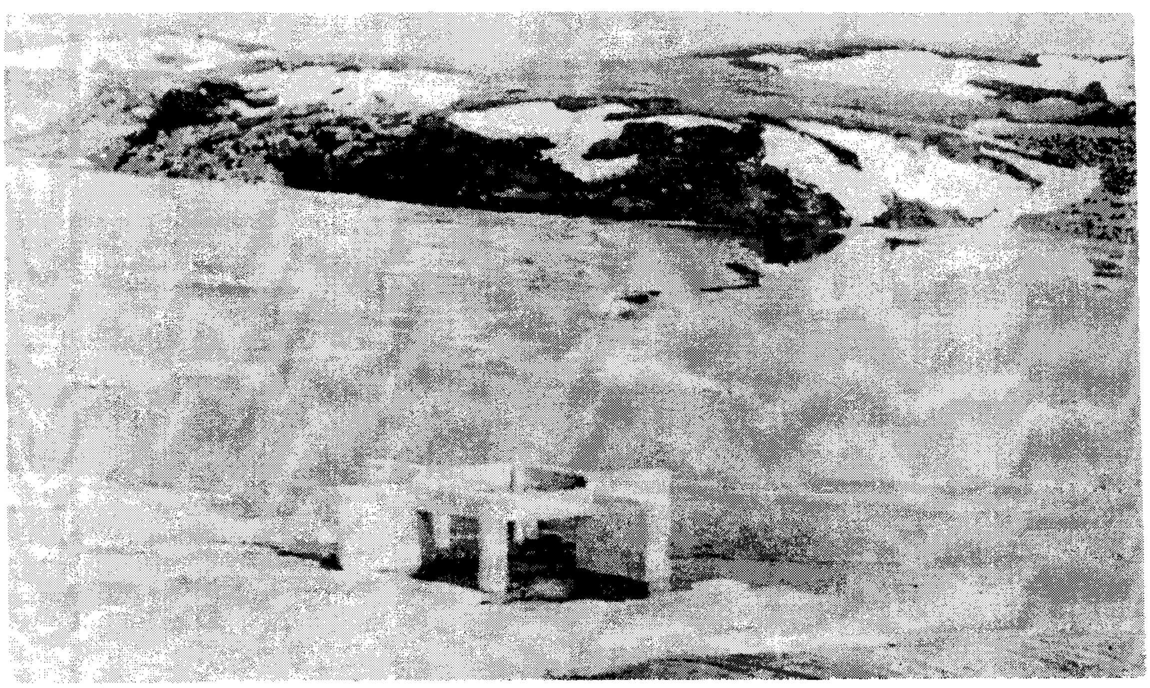
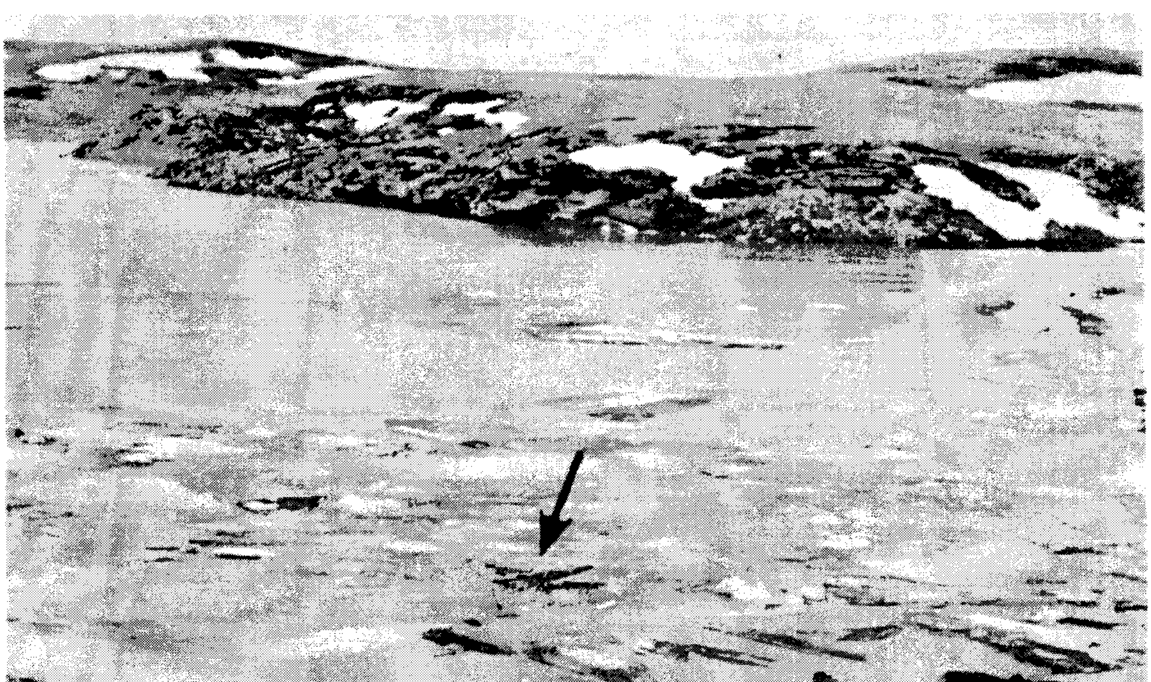


FIGURE 15--On April 17 the reservoir had risen to elevation 2073.8, submerging the spillway crest to a depth of 9.3 feet at a discharge of 3,250 cfs.

A--On April 18 the reservoir elevation was 2080.2, the head on the crest 15.7 feet, piers were submerged 1.7 feet, and the discharge was 3,650 cfs. Arrow indicates the spillway location.



B--By April 21 the reservoir had receded slightly, but water still stood a foot above the tops of the piers. Spillway at arrow.

FIGURE 16--Operation of Heart Butte spillway with piers submerged.

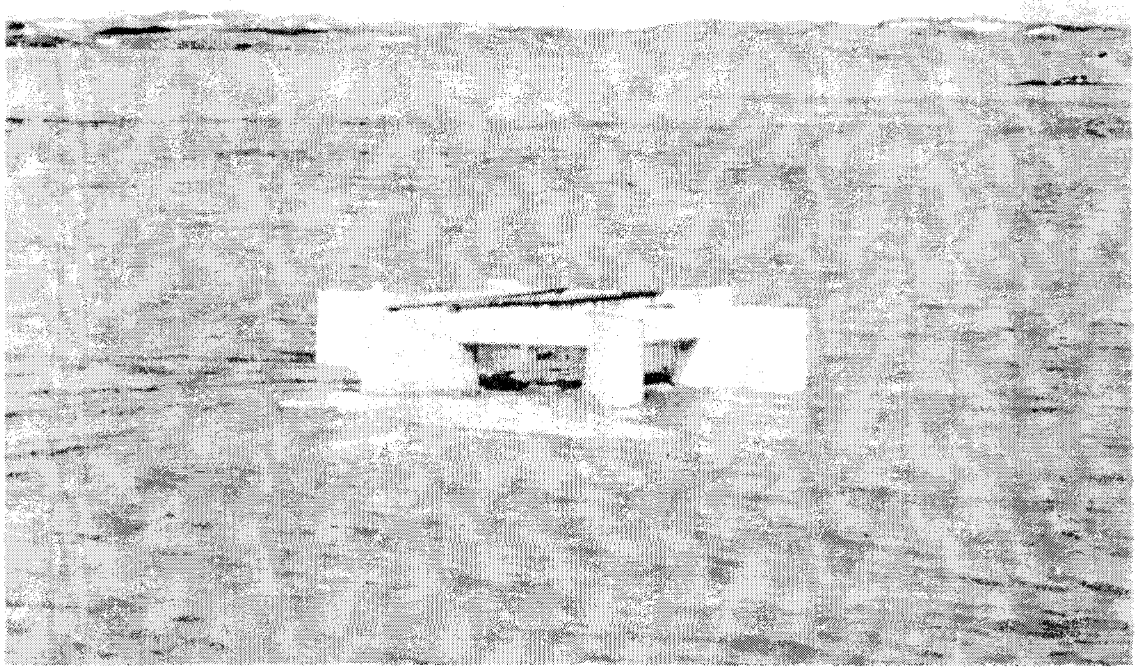


FIGURE 17--Reservoir elevation had been reduced to 2074.0 April 26. Ice hid any disturbance in the morning glory. Discharge was 3,260 cfs.

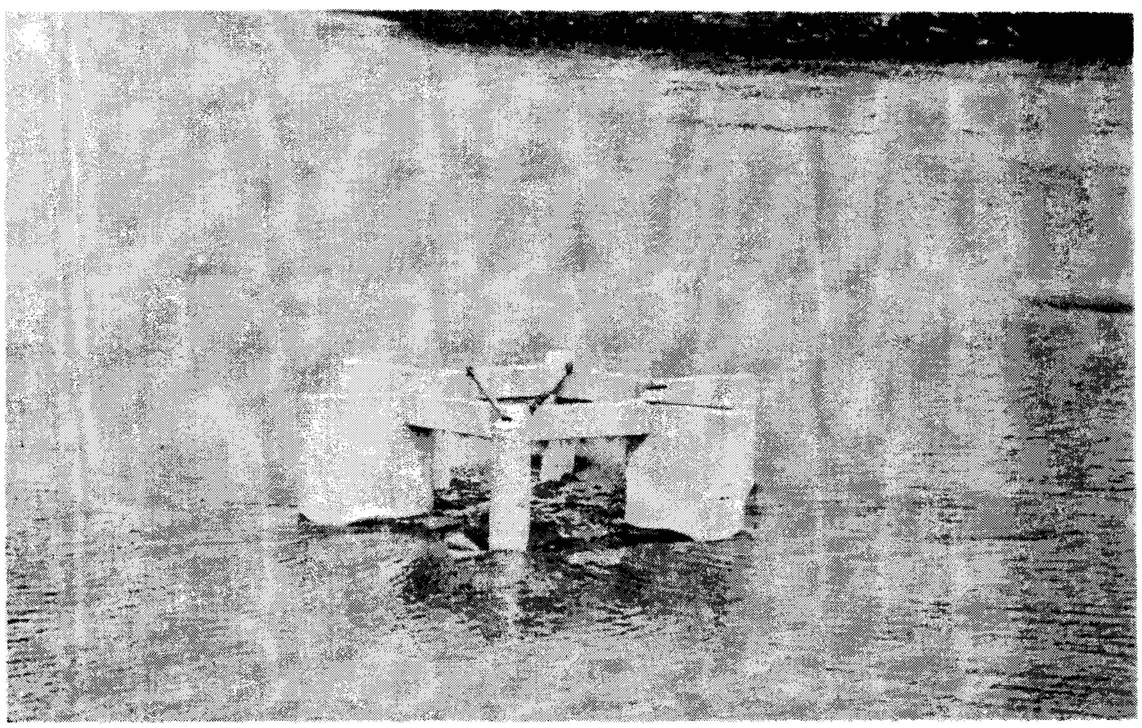


FIGURE 18--Spillway discharge was 3,090 cfs April 28, and reservoir elevation was 2071, about 0.7 foot above the point at which flow changes from submerged to free. The "mushroom" surged no more than a few inches.

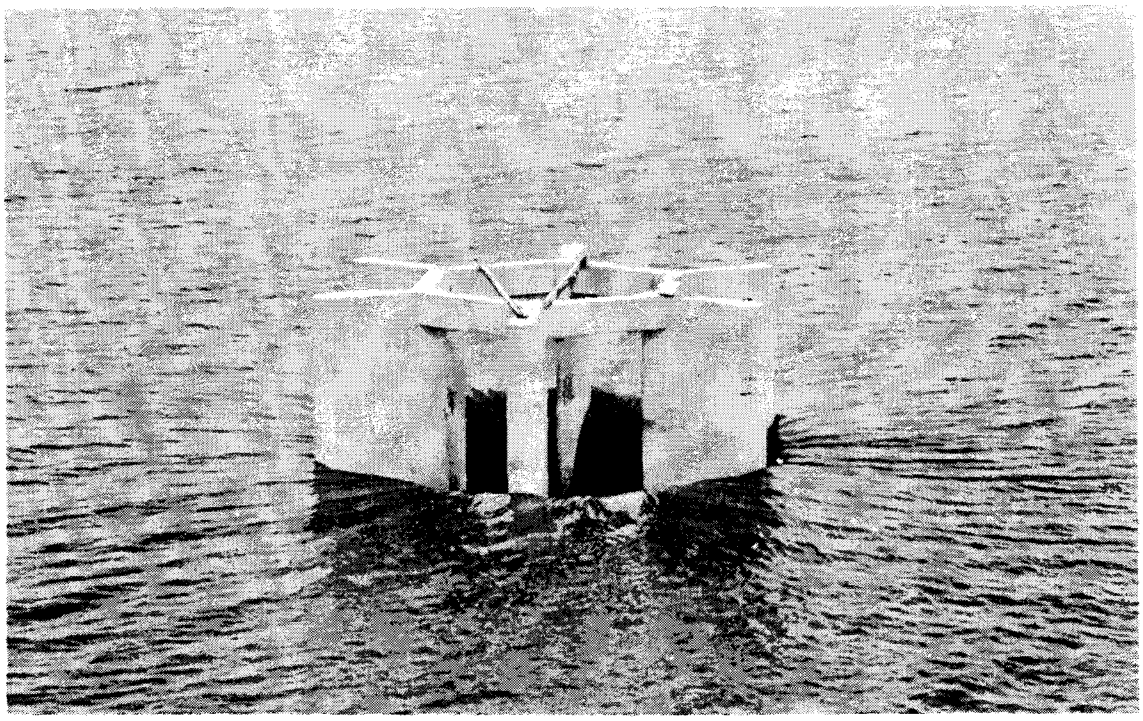


FIGURE 19--With the reservoir elevation at 2068.6 on April 30, the spillway was discharging freely at a rate of 1,600 cfs.

reservoir continued to rise throughout April 19, but on April 20 it started to recede. On April 21 the reservoir was still a foot or so above the piers, see figure 16B. The ice was breaking up fast and the wind was shifting it around the spillway area. Regardless of whether ice or water was over the spillway entrance, the operation was satisfactory, with no evidence of serious vortex action.

On April 26, with the reservoir at elevation 2074 and the piers again visible, operation was also satisfactory, see figure 17.

The reservoir was down to elevation 2071 on April 28, or about 0.7 foot above the point where the flow changes from submerged to free discharge, see figures 18 and 12. The photograph indicates the mild condition inside the spillway. There was no pulsation or rising and falling of the "mushroom."

On April 29, the reservoir had fallen to 0.8 foot below the critical submergence point and although the "mushroom" was lower in the shaft, it was still stable with no rising and falling evident. Again the flow had passed through the critical range during the night when photographs and observations were impracticable. Indications are that the prototype submerged at the headwater elevation shown by the break on the curve of figure 12 and that the change occurred abruptly as indicated on the model curve.

The spillway was discharging freely on April 30, (1,600 cfs) with reservoir elevation 2068.6, see figure 19. No spray emerged from the glory hole at this or lower heads as has occurred on some other glory hole spillways.

Throughout the flow range, no vibration was noticeable in the structure. Several excursions were made down into the outlet works gate access well while the spillway was operating. Efforts were made to detect vibration in the structure by feeling the various parts of the structure, but no vibration could be detected. Also, from a location on the top of the dam or from the reservoir banks, no "noise" could be detected. The outlet works gate was opened and closed on April 21, 1950. No noise or vibration was evident during this operation.

One year later the spillway again went into operation, reaching a maximum reser-

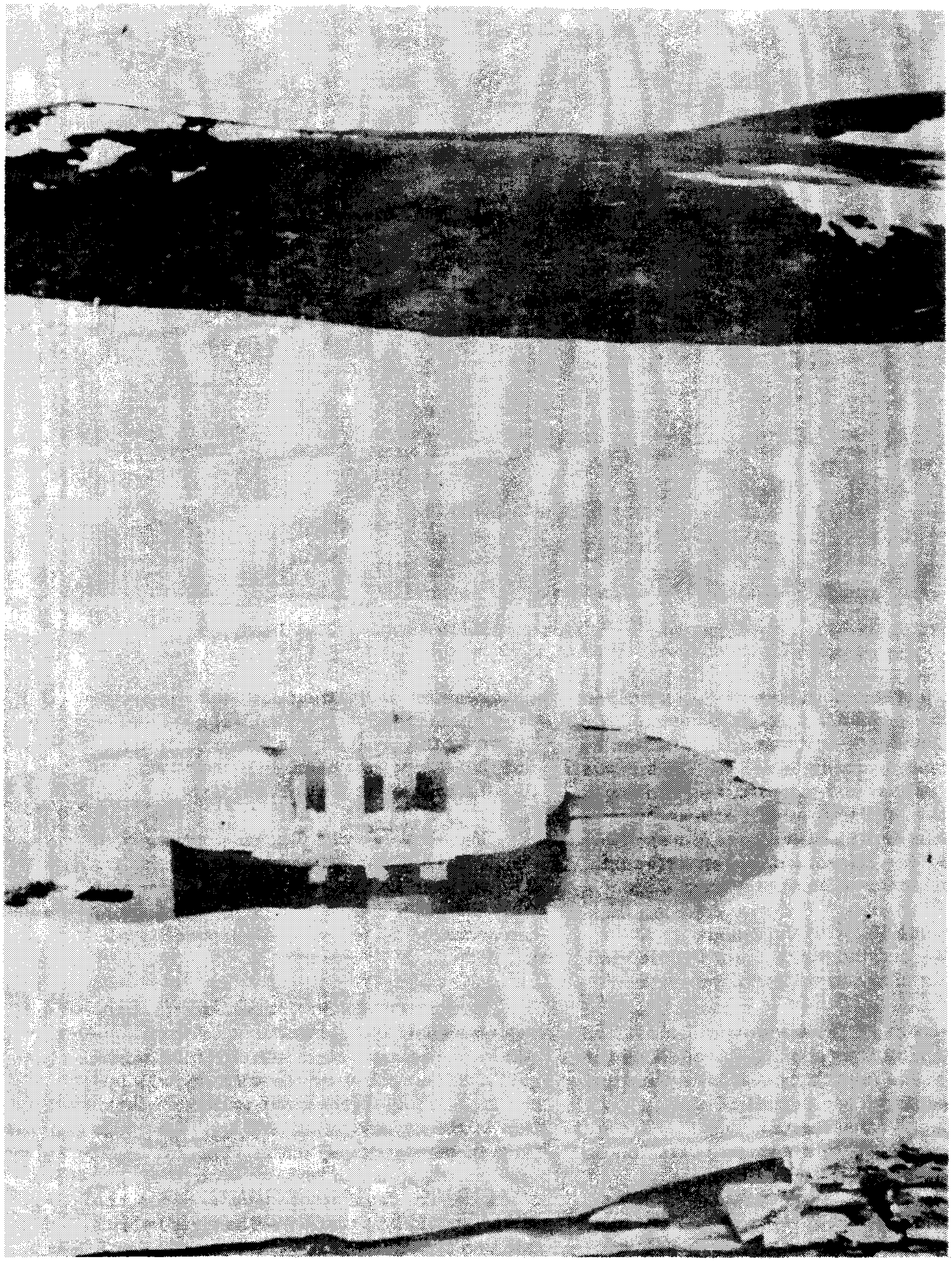


FIGURE 20--On March 27, 1951, the reservoir stood at elevation 2070.0, about 0.2 foot below the submergence point. Ice thickness of 36 inches in February had caused no operating difficulties. Discharge was 2,650 cfs.

voir elevation of 2075, or about 7 feet less than occurred in 1950. On March 27, 1951, the reservoir was at elevation 2070, see figure 20, about 0.2 foot below the submergence point. Again the operation was satisfactory with no visible difficulty despite the fact that on February 15, 1951, the ice in the reservoir was 36 inches thick.

3. Spillway air demand.--Measurements were made in the model to determine the quantity of air being entrained by the spillway discharge as it passed over the air-entraining deflectors located on the spillway face just below the spillway crest, see figure 5. Air-flow measurements in the model were made using a 3/8-inch-diameter sharp-edged orifice connected to a differential water manometer. All air entering the model passed through the orifice before entering the venting system. Since the differential was extremely small for the air quantity flowing in the model, a specially constructed gage was used which multiplied the actual differential so that more consistent readings could be obtained throughout a series of tests. The gage was calibrated to provide reasonably accurate air measurements, but consistency was considered more important than absolute accuracy.

At the time of prototype construction, pipe was extremely difficult to obtain on short notice. Since the model tests continued throughout most of the construction period, only a small amount of pipe and special fittings could be provided for measuring stations in the prototype. Thus, the data obtained from the prototype are not sufficient to determine pressures in various parts of the venting system, but do indicate the quantity of air flow in the prototype for various spillway discharges.

The air quantity flowing in the prototype vents was determined by measuring the air velocity with an anemometer hand-held in the 18-inch-diameter air vent pipe. Air velocity determinations were made in one of the vertical pipes contained in the wall of the gate operating house and in one of the horizontal pipes just upstream from the point where it emerges into the tunnel junction section, see

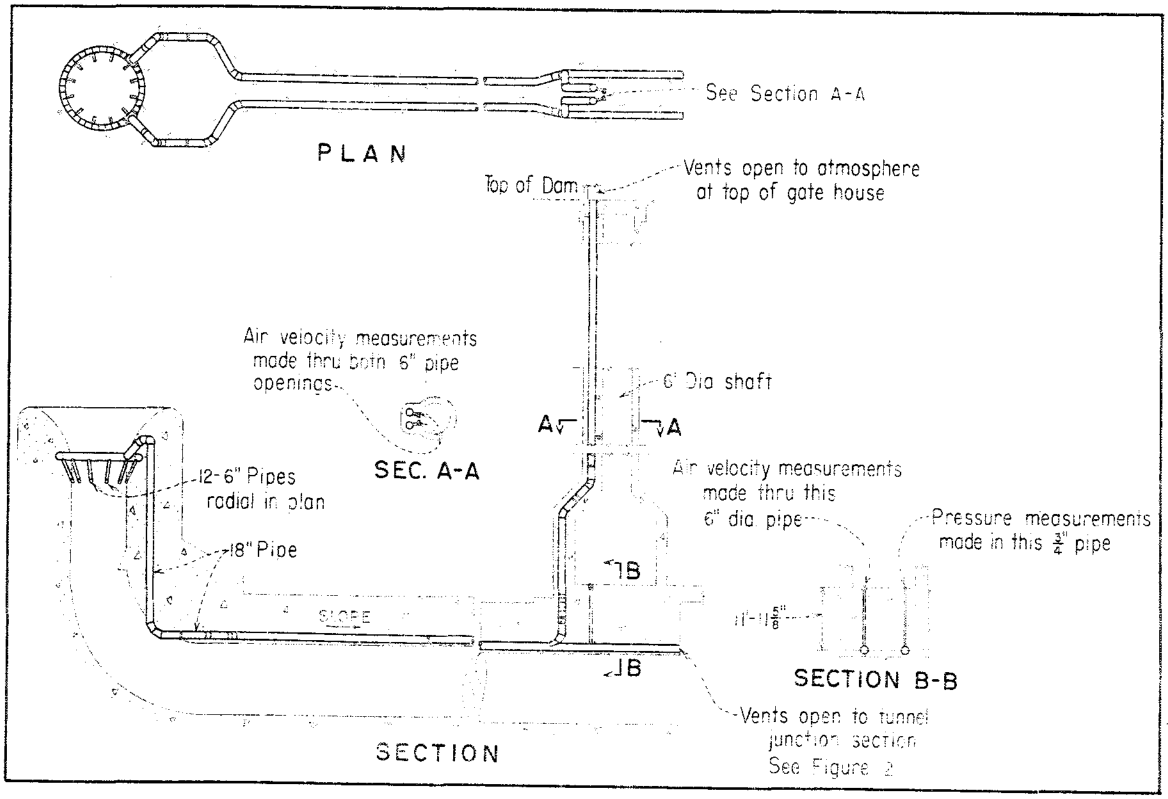


FIGURE 21--Spillway air vent system at Heart Butte Dam.

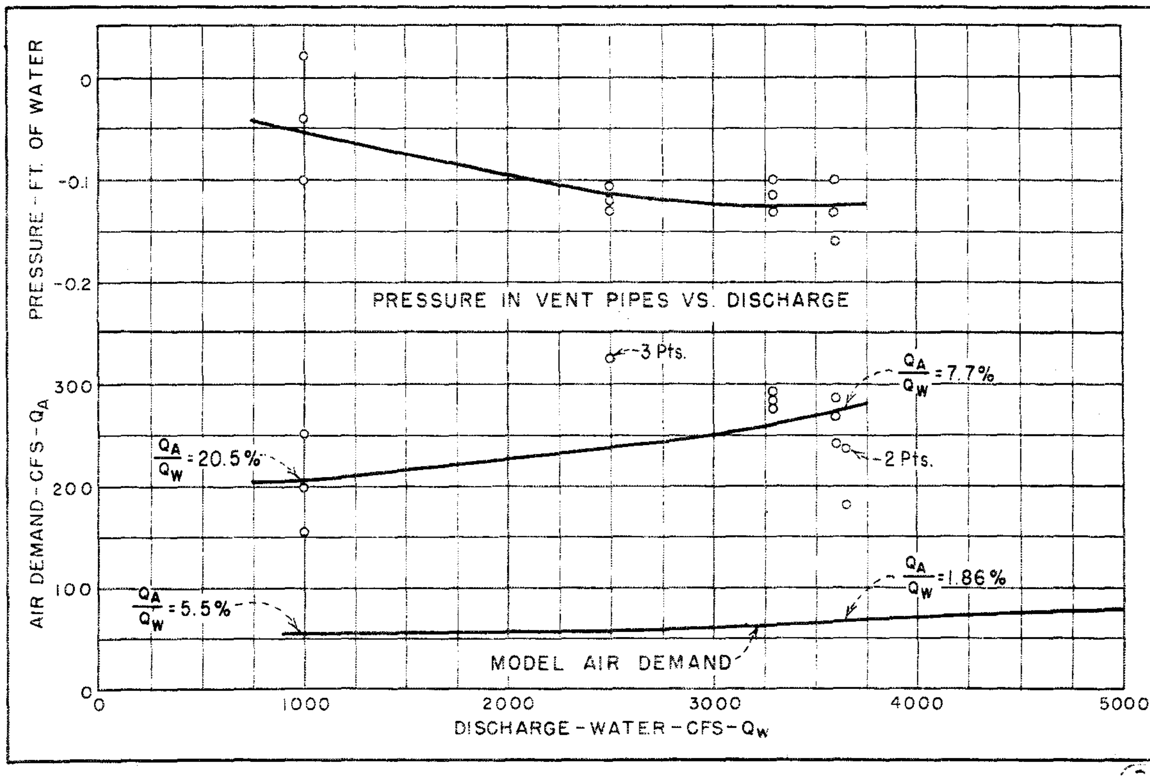


FIGURE 22—Air demand curves for model and prototype for varying discharges.

figure 21. Concurrent with the air-velocity measurements, pressure measurements were made on the other horizontal air vent using a U-tube containing water for an indicator. The pressure-measuring station is also shown in figure 21.

Air flow in the prototype was not smooth, as evidenced by the sound of the air flow and from the difficulty experienced in holding the anemometer steady. There was chance for considerable error in any one anemometer measurement and so several determinations were made for each flow in both the vertical and horizontal vent pipes. Readings were taken until the observer was satisfied that a true average had been obtained; a consistent set of readings over a period of 10 or more minutes was obtained. The anemometer recorded lineal feet of flow which, when divided by the elapsed time, gave the air velocity in feet per second. Each observation lasted about 2 minutes so that the average velocity of air flow was that occurring for a testing time of 6 to 12 minutes. Pressures measured in the U-tube also indicated that the air flow was not steady. Differentials varied from plus to minus, but an average pressure reading was easier to obtain than was the air velocity.

The results of the air quantity and pressure determinations are plotted on figure 22. The percentage of air entrained in the spillway discharge, for both model and prototype, showed a decrease as the discharge increased. In this respect the model predicted the performance of the prototype. The prototype, however, entrained roughly four times as much air as was predicted by the model. In this respect, also, the prototype performed as anticipated except that accurate predictions of how much more air the prototype would entrain could not be made from the model tests. Where the model showed air entrainment of 5.5 percent of the water discharge for 1,000 second feet of spillway discharge, the prototype showed 20.5 percent. For 3,600 second feet the model showed 1.9 percent and the prototype 7.7 percent.

The points from which the curves of figure 22 were drawn are also shown in the figure. The prototype air demand curve was not drawn through the points for 2,500 second feet because the pressure values, which were considered more reliable, indicated that the curve should be drawn below the velocity points. Moreover, the shape of the curve was then similar to the model curve which was based on very consistent data. To further prove the validity of the shape and values of the prototype air demand curve, computations of air flow were made using the measured pressures, assuming that both vent pipes carried equal quantities of air and using the usual losses for bends, friction, inlet, etc. The computed values were found to be in fair agreement with those shown in figure 22.

4. Stilling basin performance.--The performance of the stilling basin was satisfactory in every respect and, furthermore, it performed according to the predictions made from the model tests. A general view of the basin and surrounding area is shown in figure 23.

Water leaving the tunnel appeared to be fully aerated and at the approximate depth indicated in the model studies, see figure 24A. The entire basin contained extremely turbulent water, see figure 24B, and was long enough to obtain the full jump height before the flow entered the excavated channel, see figures 25A and 25B. At times considerable amounts of spray shot into the air, at a point where the outflow from the tunnel plunged beneath the tail water but most of the spray fell back into the basin. The small amount of spray which fell adjacent to the basin caused no difficulty. Much of the time the flow entered the basin smoothly as shown in figure 25A. Flow leaving the basin had a relatively quiet water surface with few measureable waves, figure 25B. There were long period swells, however, with a maximum height of 12 to 18 inches which were caused by pulsation in the stilling basin. The disturbances below the stilling basin were similar to those noted during the model tests.

Water-surface profiles were measured in the prototype for discharges of 3,700, 3,300, 2,350, and 1,050 cfs. These are shown in figures 26 and 27, along with the profile obtained during the model tests for 5,600 cfs. Although no exact comparisons can be made, the prototype profiles seem to be in good agreement with the model profile. If differences do exist, they are probably due to the greater air entrainment in the prototype.

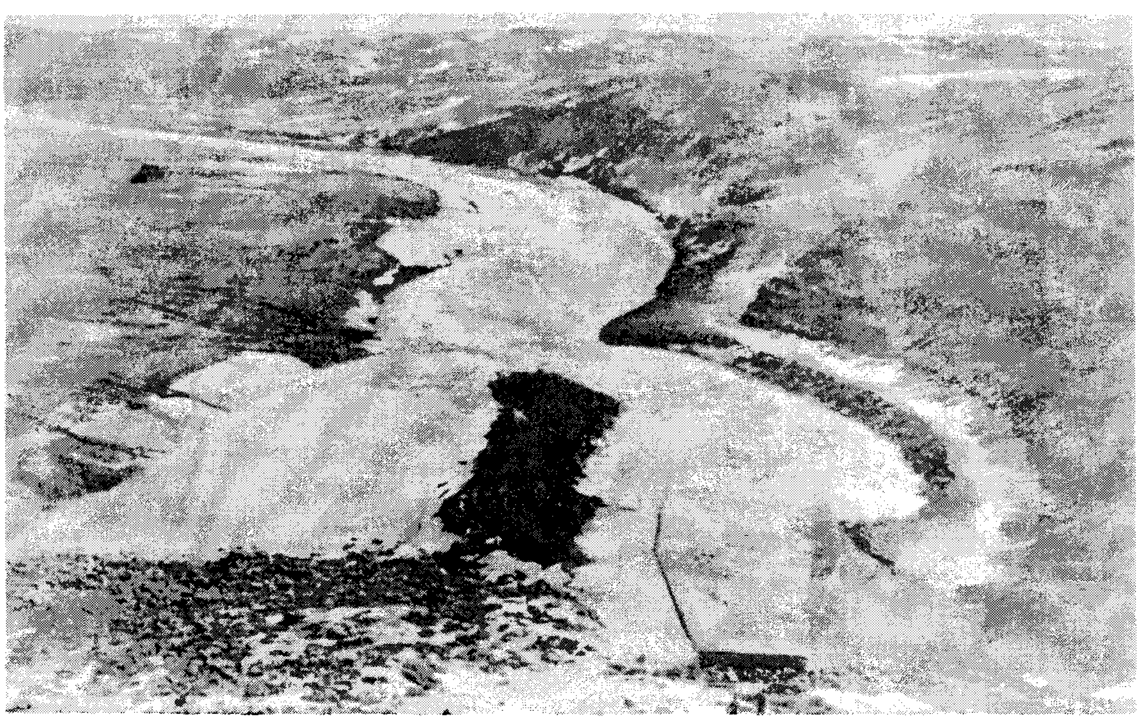
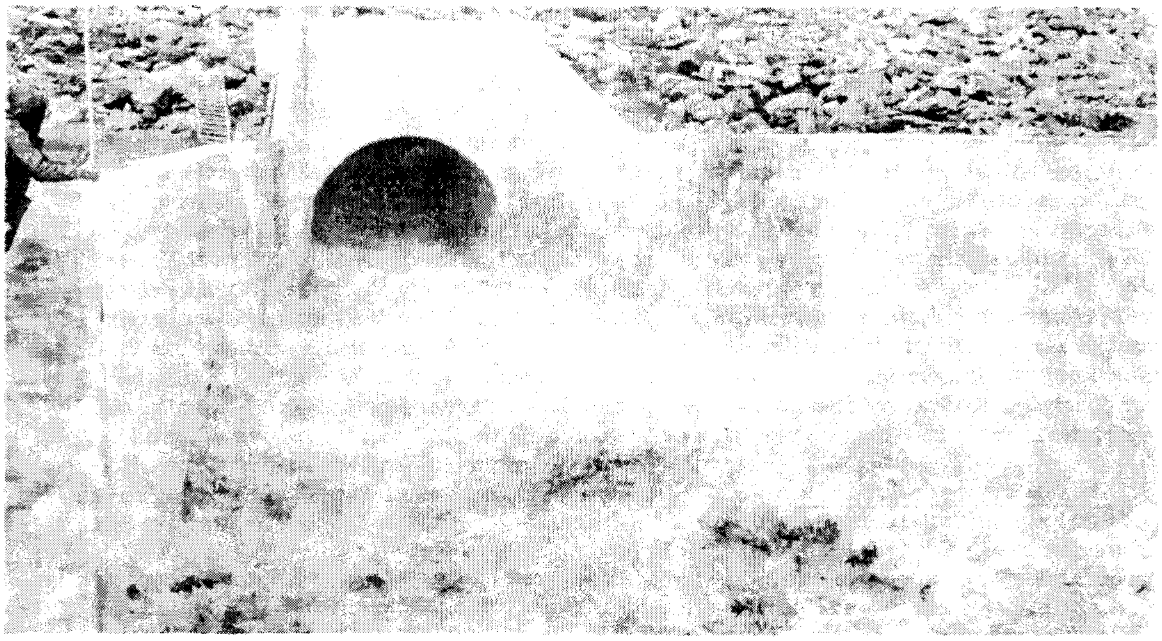
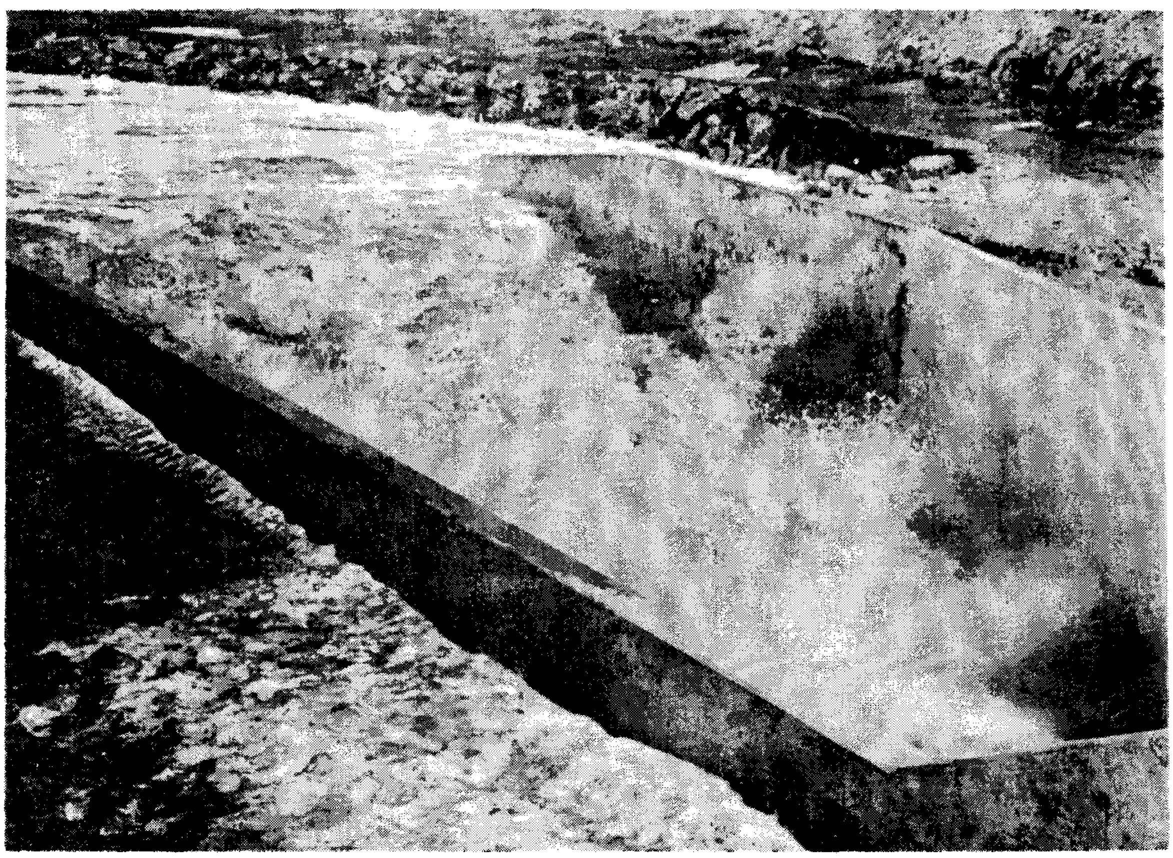


FIGURE 23--Outflow at 3,250 cfs, about 58 percent of capacity. The stilling basin effectively dissipates the energy of the discharge.



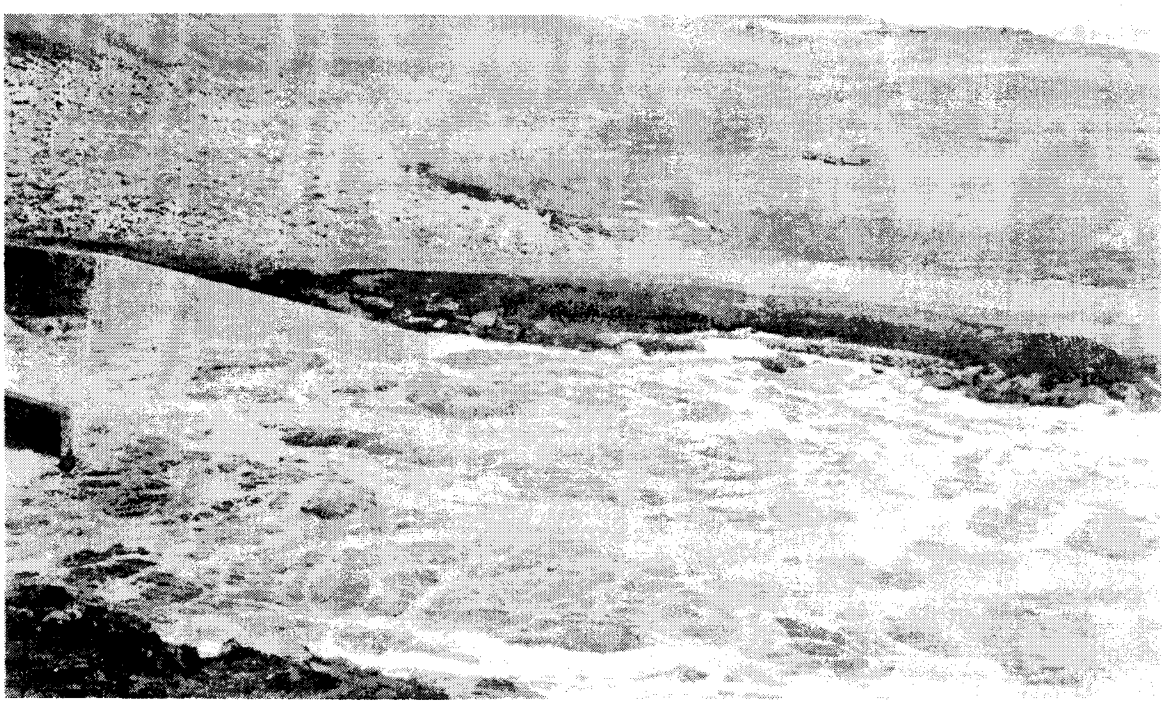
A--Flow entering the basin was well aerated and well distributed across the basin width.



B--Profile of the hydraulic jump showed along the wall. Flow leaving the basin was smooth and uniform.

FIGURE 24--Performance of the stilling basin at a discharge of 3,600 cfs.

A--Dividing walls, submerged here, were effective in spreading flow to entire basin width.



B--Flow leaving the basin had swells and boils, but no choppy waves.

FIGURE 25--Performance of the stilling basin at Heart Butte Dam in dissipating energy of discharge at a flow of 3,600 cfs.

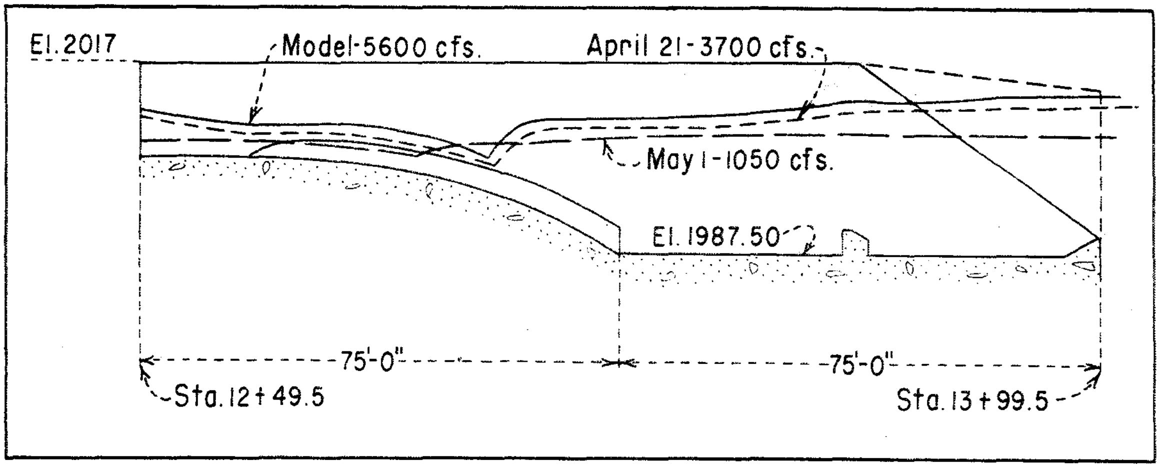


FIGURE 26--Profiles of comparative discharges of model and prototype spillway.

This would make the prototype profiles slightly higher than those in the model for the same discharge.

5. Erosion downstream from stilling basin.--Erosion tests in the model indicated that the channel banks just downstream from the stilling basin would be subjected to greater erosion forces than the channel bottom and that rock riprap would be necessary in the prototype to prevent bank damage. The channel bottom was shown by the model tests to be relatively free from erosion tendencies and no damaging erosion was expected there. As a precaution, however, because of the fine-textured friable material composing the channel, rock riprap was used in the prototype channel bottom. No riprap was used in the model tests.

Before the run-off in the spring of 1950, cross sections had been taken in the prototype channel on May 31, 1949. Following the 1950 run-off, cross sections were again taken on June 15, 1950. Cross sections for both dates at a station located just downstream from the end sill and at a station 50 feet downstream from the sill are shown in figure 28. These typical sections show the maximum erosion depth to be less than 12 inches. Close to the end sill there is no significant erosion. Using all the cross sections taken, see figures 28 and 29, calculations indicated that less than 20 cubic yards of material was

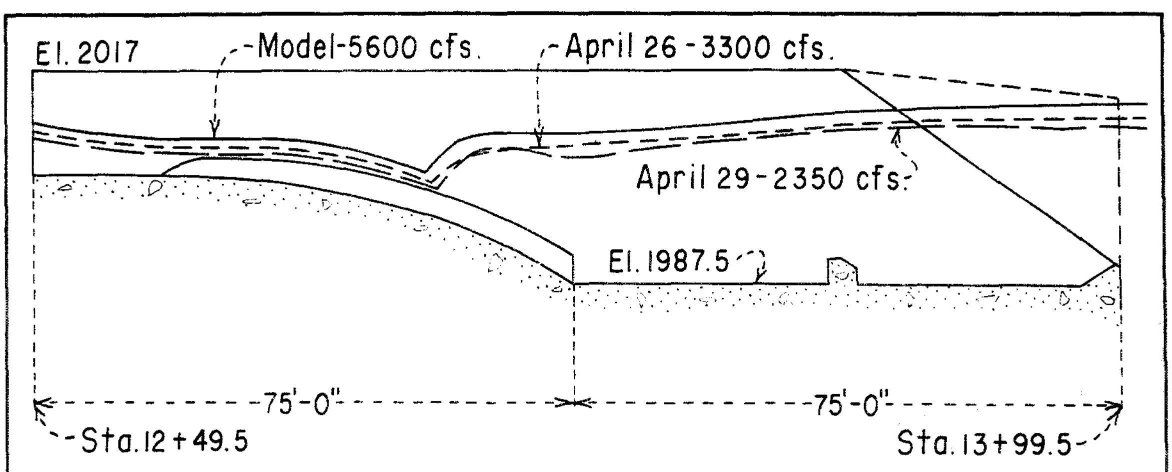


FIGURE 27--Profiles of comparative discharges of model and prototype spillway.

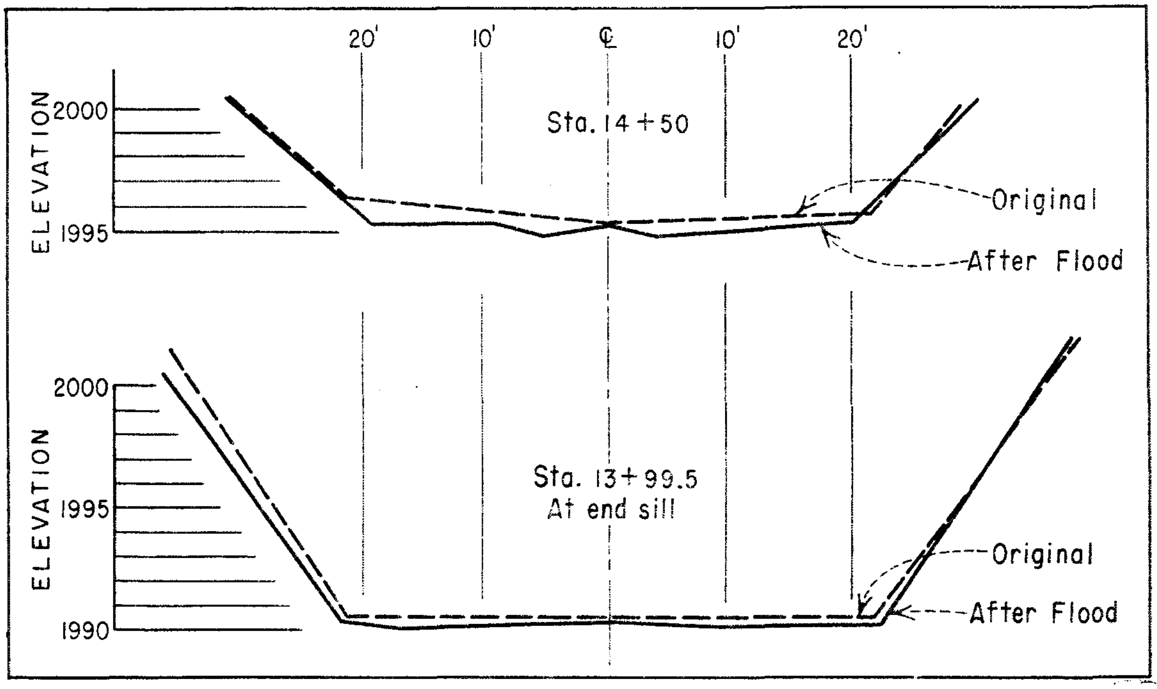


FIGURE 28--Comparative cross sections show that there was little erosion in the river channel below the stilling basin during the 1950 flood.

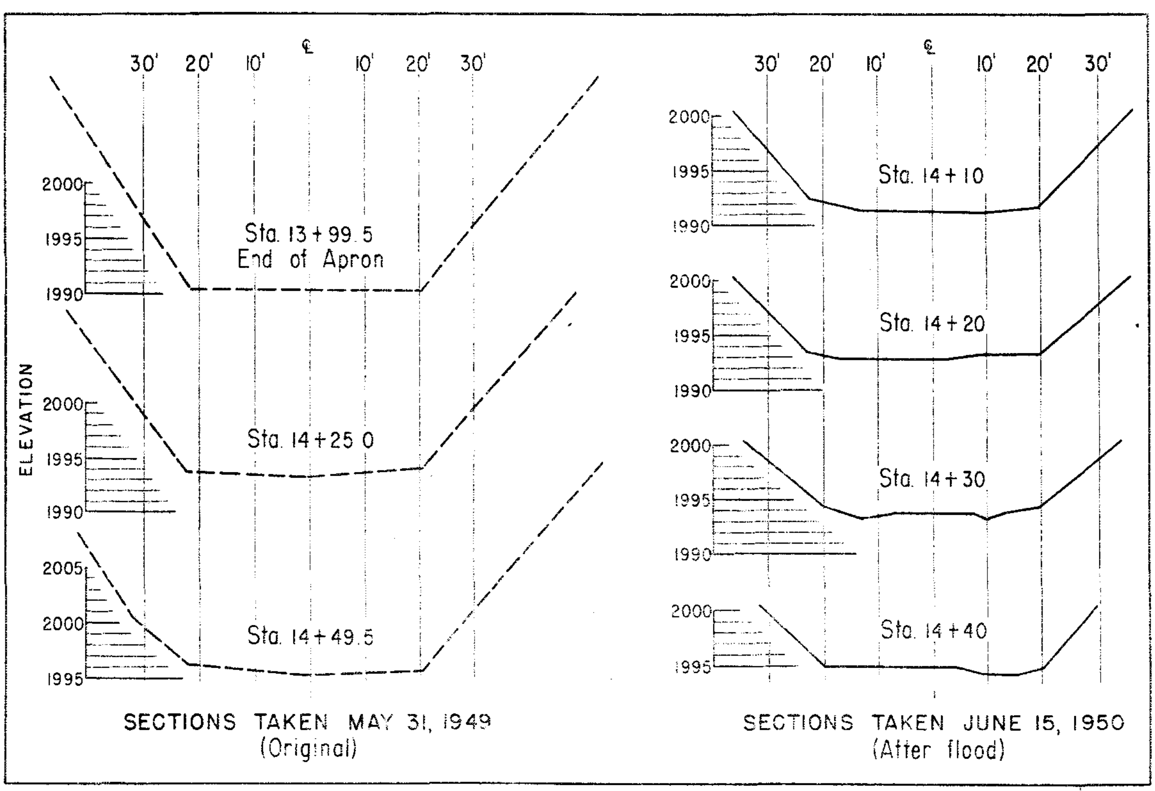


FIGURE 29--Detail cross sections showing before and after conditions in river channel below the stilling basin.

A--Swells eroded fine material from behind the coarse riprap along the left bank of the excavated channel below the stilling basin.

B--Erosion also occurred along the right bank near the junction of the natural river channel and the excavated channel below the basin.

FIGURE 30--Erosion of the excavated channel downstream from the stilling basin.

eroded and removed from the channel bottom during the entire run-off.

The channel banks, despite their riprap cover, were eroded to a greater degree. The riprap, however, had been placed in a thin layer, was not well graded as to size, and in places the earth banks could be seen between the individual rocks. Swells were observed to rise over local areas and penetrate very easily into the large voids. When the water receded, some of the earth was removed from behind the riprap. This was evidenced by the darker, earth-colored water which could be seen adjacent to the riprap. After several days of spillway operation, the riprap had slumped and the earth banks had caved as shown in figure 30. In spite of the apparent damage to the banks the riprap still continued to provide a good measure of protection against further cutting.

The bank damage was caused primarily by swells originating from surges in the hydraulic jump, and not by waves of the ordinary variety, since these were only a matter of inches in over-all height. The model stilling basin had been equipped with baffle piers and chute blocks to reduce the over-all length of the stilling basin and decrease its cost. It had been noted during these and other model tests that when a hydraulic jump is reduced in length by the use of artificial devices such as baffle piers that the jump becomes more stable in most respects, but it does exhibit a tendency to produce the swells discussed above. The swells are considered the lesser of the evils, however, and are not impossible to cope with. With a thicker application of well-graded riprap, there probably would have been no damage.

6. <u>Tail-water elevations</u>.--The topography in the model tail box included the excavated channel but did not include any portion of the Heart River Channel, see figure 9. Tail-water elevations were set by means of an adjustable tail gate located at the end of the model using a computed curve, tail-water elevation versus discharge, for the Heart River. The excavated channel was designed so that the tail-water elevation in it would be essentially the same as that to be expected in the river channel. The tail-water curve used in the model tests and shown in figure 31 was computed for a point located

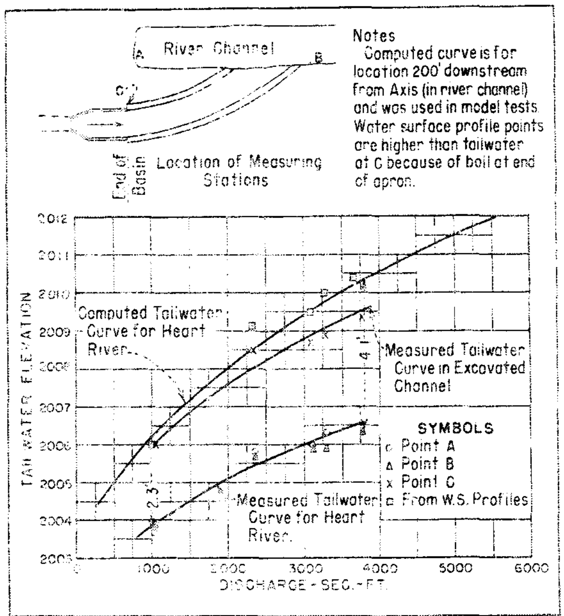
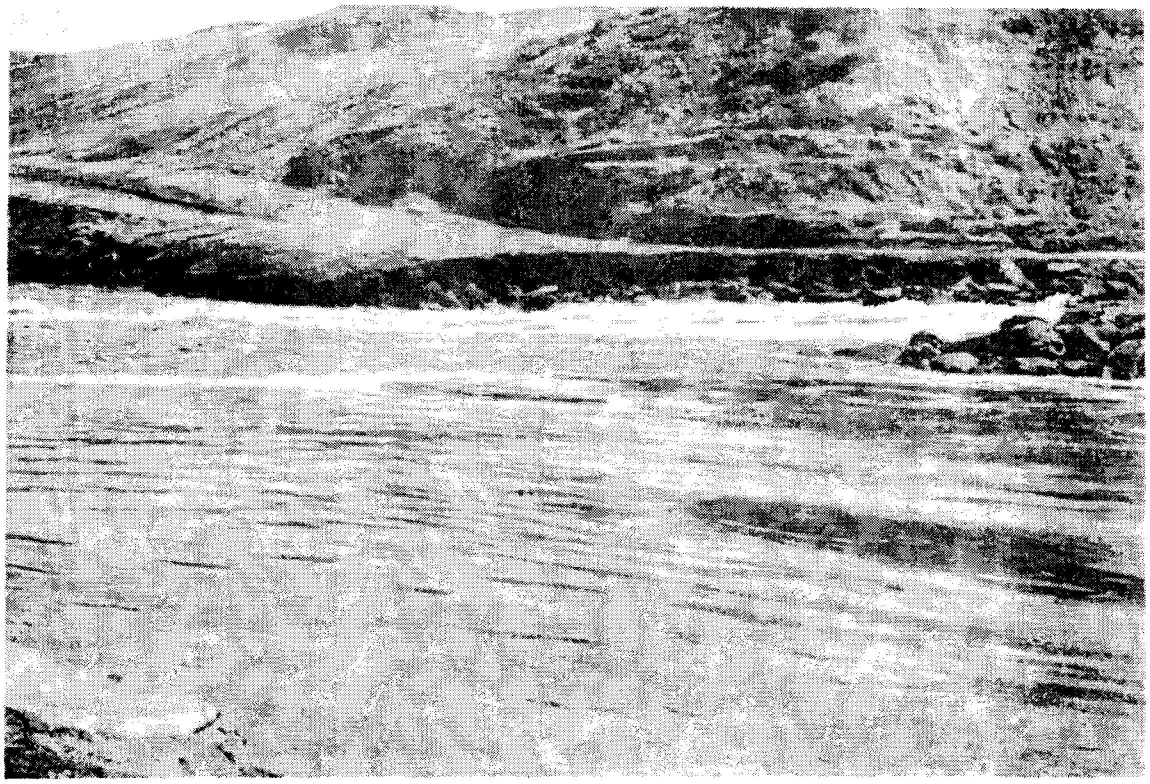


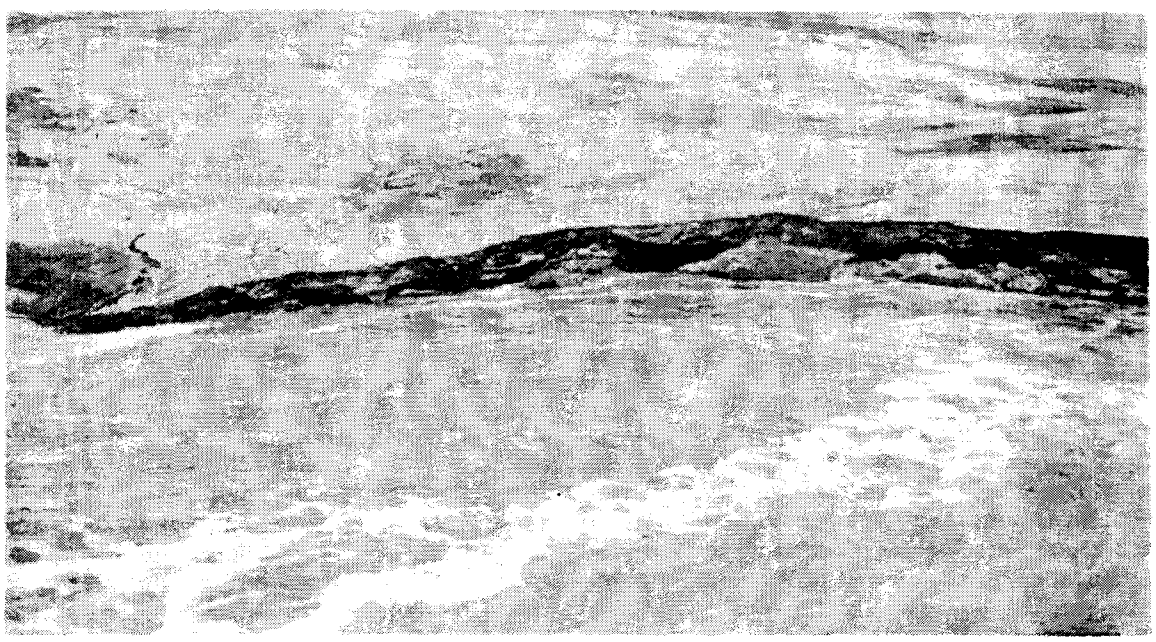
FIGURE 31--Comparison of measured and computed tail water elevations.

200 feet downstream from the axis of the dam in the Heart River.

During the prototype operation it was readily apparent that the tail-water elevation in the river was considerably lower than that in the excavated channel. Water entering the river from the channel had a steep surface slope and a much higher velocity than anticipated, see figure 32A. Observations, however, were not sufficient to establish whether the tail water in the channel was too high or that in the river too low. Levels were run to determine the tail-water elevation at four separated points for five different discharges. The location of these points, together with the tail-water elevation and discharge, is shown in figure 31. These values are plotted below the tail-water curve used in the model tests. These data show the computed tail-water curve to be 2.3 feet higher at 1,000 second feet and 4.1 feet higher at 3,600 second feet than the actual measured points in the Heart River. Tail-water elevations measured in the excavated channel more nearly coincided with the computed curve, but at 3,600 second feet the elevation at Point C, figure 31, taken in a quiet area adjacent to the wing wall at the end of the apron, was 1 foot below the computed curve. Elevations obtained from water-surface profiles taken along the basin center line agreed with the computed curve, but only because they in-



A--The 4-foot difference in elevation between the flow in the excavated and natural channels at 3,700 cfs accelerated the discharge.



B--The high-velocity discharge into the natural channel set up a back eddy that damaged the river bank upstream from the point of impact.

FIGURE 32--Flow conditions at the junction of the excavated and natural channels.

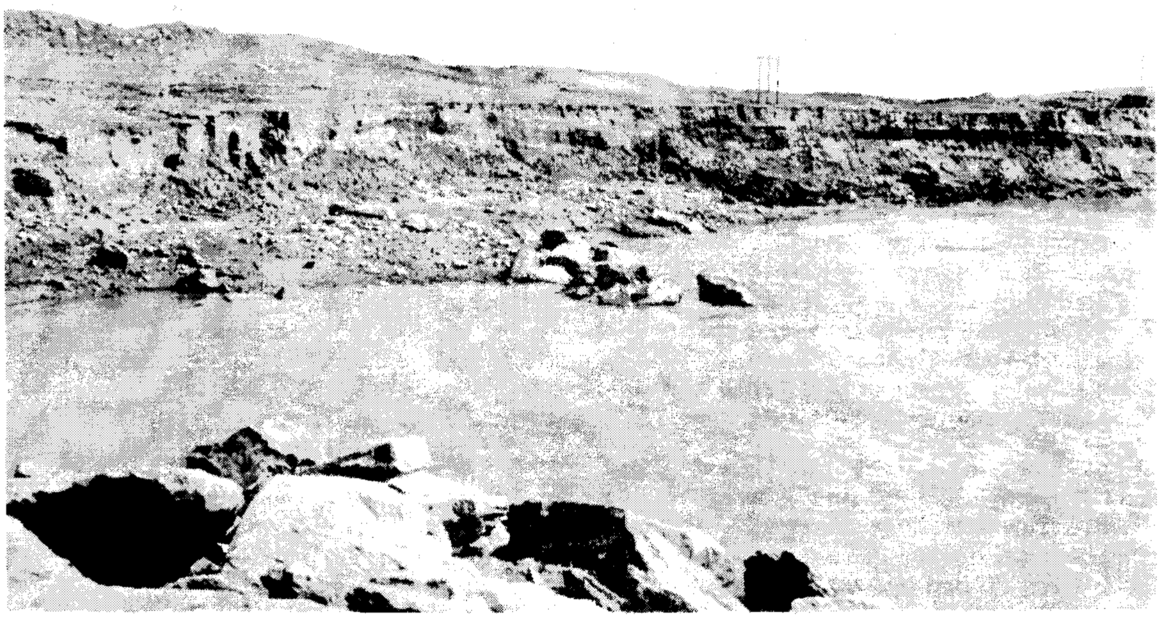


FIGURE 33--Loss of bank material through erosion during the 1950 flood is readily apparent. The eddy shown in figure 32-B caused this damage.

cluded the boil height at the end of the apron, which was slightly higher than the adjacent tail water.

The model stilling basin was tested to determine the permissible reduction in tail water before the jump would be swept off the apron. In the model it was possible to lower the tail water only 3 to 4 feet before the jump was swept out for the maximum discharge of 5,600 second feet. Since the tail-water elevation in the Heart River is 4.1 feet lower, at a discharge of 3,600 second feet, than the computed tail water, it is imperative that a close watch be kept on the excavated channel to prevent damage which might lower the tail water to the level of the Heart River. If this should happen the jump will undoubtedly sweep out and the apron will operate as a flip bucket. Since the structure is not designed for this type of operation, damage could result.

7. River erosion.--The difference in water levels between the excavated channel and the river was the cause of the high-velocity flow entering the Heart River. Water leaving the stilling basin was of relatively low velocity and would not have caused ill effects as it entered the river, see figure 25B. The 4-foot difference in elevation, however, caused an increase in velocity which proved to be sufficient to cause considerable damage to the unprotected riverbank downstream, see figure 32B. Some of the damage was caused by the direct effects of the current flowing diagonally across the river and cutting into the far or left bank. A great share of the damage, however, was caused by a large induced eddy in the river, see figure 32B. This eddy caused an upstream current along the left bank which removed large volumes of material from areas considerably upstream from the point where the main flow impinged on the bank. Although the damage was considerable in extent, it had no ill effects on the structure or its operation. Riprap placed in the eroded area will prove of value, however, since the damage would become greater with each successive run-off and the end result would be difficult to predict. The bank damage is illustrated in figure 33. A comparison of figures 23 and 34 shows the extent of the bank damage which occurred between the start of the run-off and May 5, 1950.

8. Inspection of structure following 1950 and 1951 floods.--An inspection of the spillway conduit was made following the 1950 flood and again following the 1951 flood. Certain findings are of interest and are discussed herein.

The conduit inspection, in both instances, revealed that the concrete was in excellent condition with the exception of four small eroded areas located in the 90° bend. Following the 1950 flood, plaster casts were made of the two most prominent areas. These are shown about full size in figures 35 and 36. The largest area is about the size of a man's hand and by actual measurement has a maximum depth of erosion of 3/4 inch. The smaller area shows a maximum depth of 1/2 inch. The surfaces shown in figures 35 and 36 were molded in sponge rubber against the plaster casts made in the field and are therefore an exact replica of the tunnel surface following the 1950 flood.

These areas are located near the invert and near the bottom of the 90° bend. Construction timbers or ice falling into the shaft could have caused the surface damage shown. Persons who have viewed the rubber casts have been of the unanimous opinion that the damage was not started by cavitation.

Following the 1951 flood, these areas were again noted. There did not appear to be any marked change in these areas as a result of the 1951 spring floods. No repairs were believed necessary.

Inspection of the riprap downstream from the stilling basin, following the 1950 flood, indicated that repairs would be advisable. The slumped riprap in the channel immediately downstream from the stilling basin structure was repaired in May and June 1950. Gravel backfill was placed on the slopes to bring them to grade and rock replaced over the gravel. The eroded riverbank just downstream from the end of the riprapped channel was sloped and covered with gravel and rock.

SHADEHILL DAM STUDIES

Description of the Project

Shadehill Dam is a part of the Missouri Basin Project and is located near Lemmon, South Dakota, on the Grand River, a tributary of the Missouri. Figure 1 is a map of the general area. The dam was constructed to provide irrigation and flood control benefits and is similar in many ways to Heart Butte Dam.

Shadehill Dam is a compacted earth-fill structure, rising 125 feet above the stream bed to elevation 2318. At normal water surface elevation the reservoir contains about

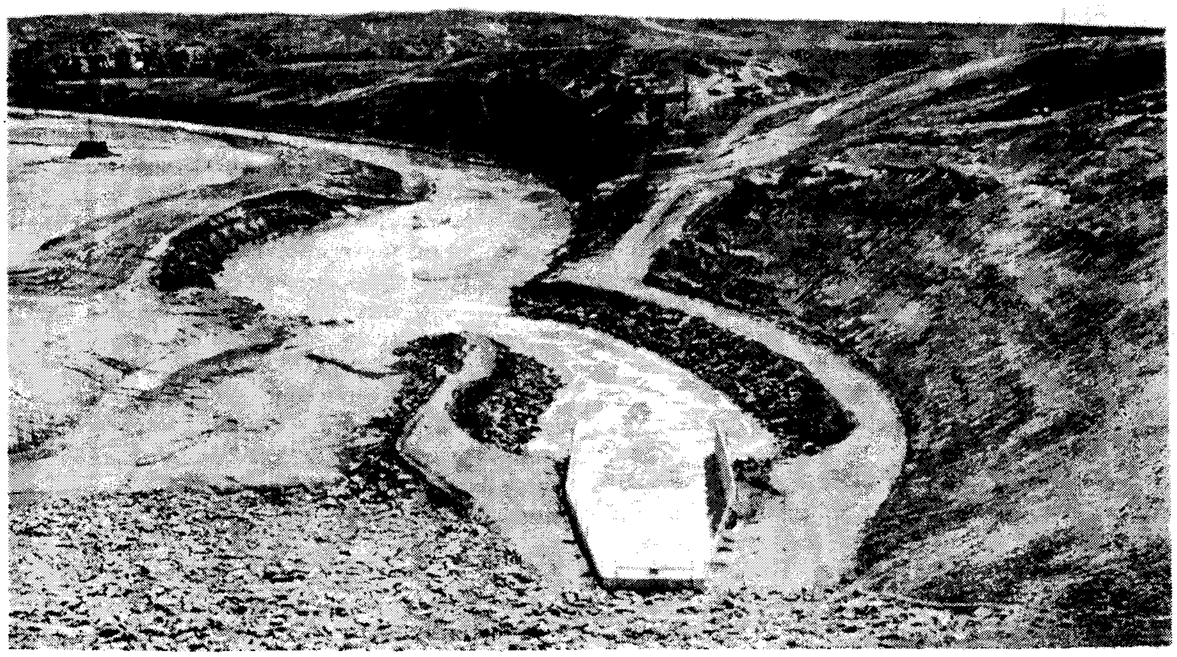


FIGURE 34--Extent of downstream erosion can be seen by comparing this view after the flood had receded with figure 23, at the height of the flood flow.

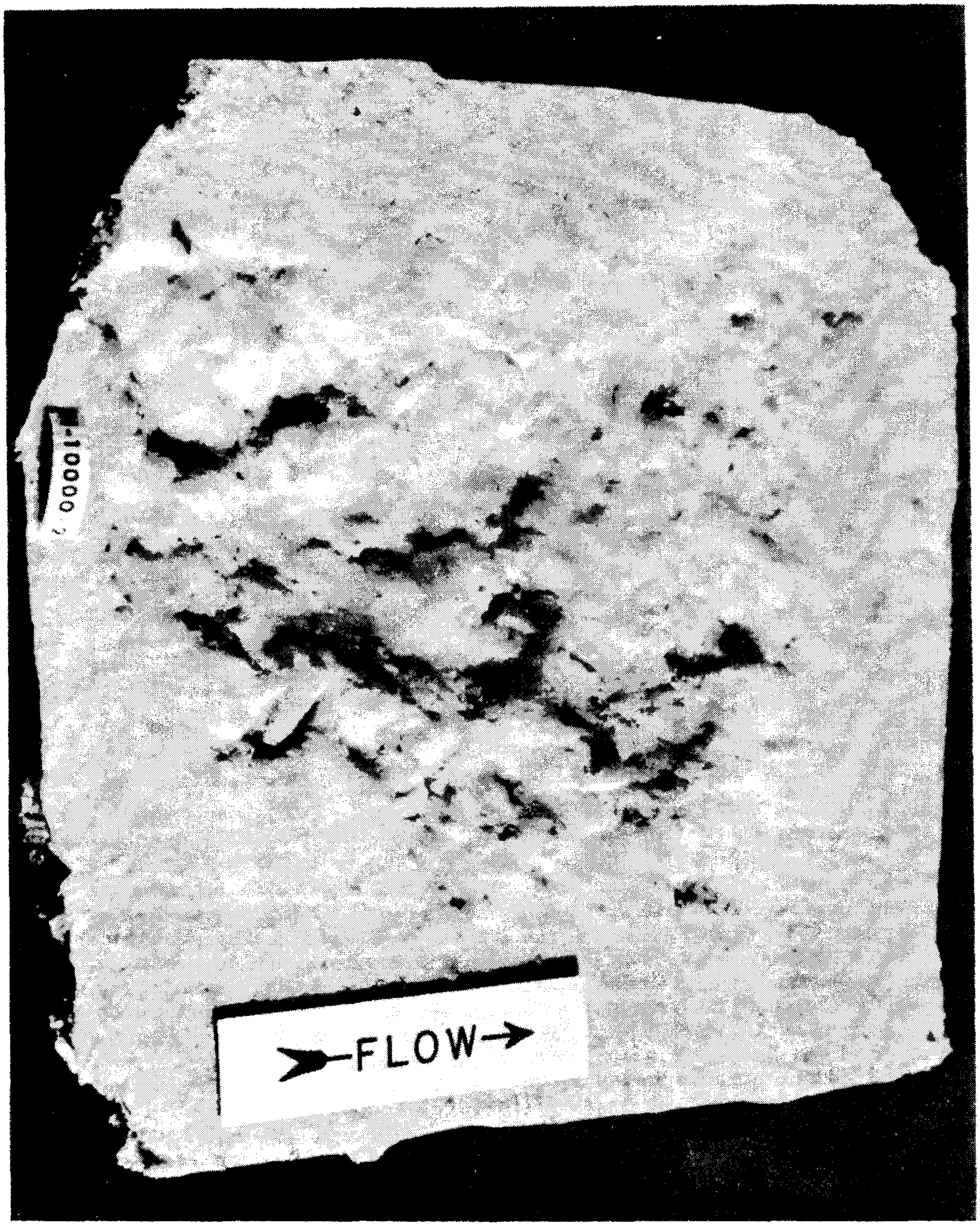


FIGURE 35--Eroded areas in the 90° bend subsequent to the 1950 flood were four in number. This, the deepest, is shown by a cast made in the bend from the actual depression. Erosion was 3/4 inch deep. The illustration is full size.

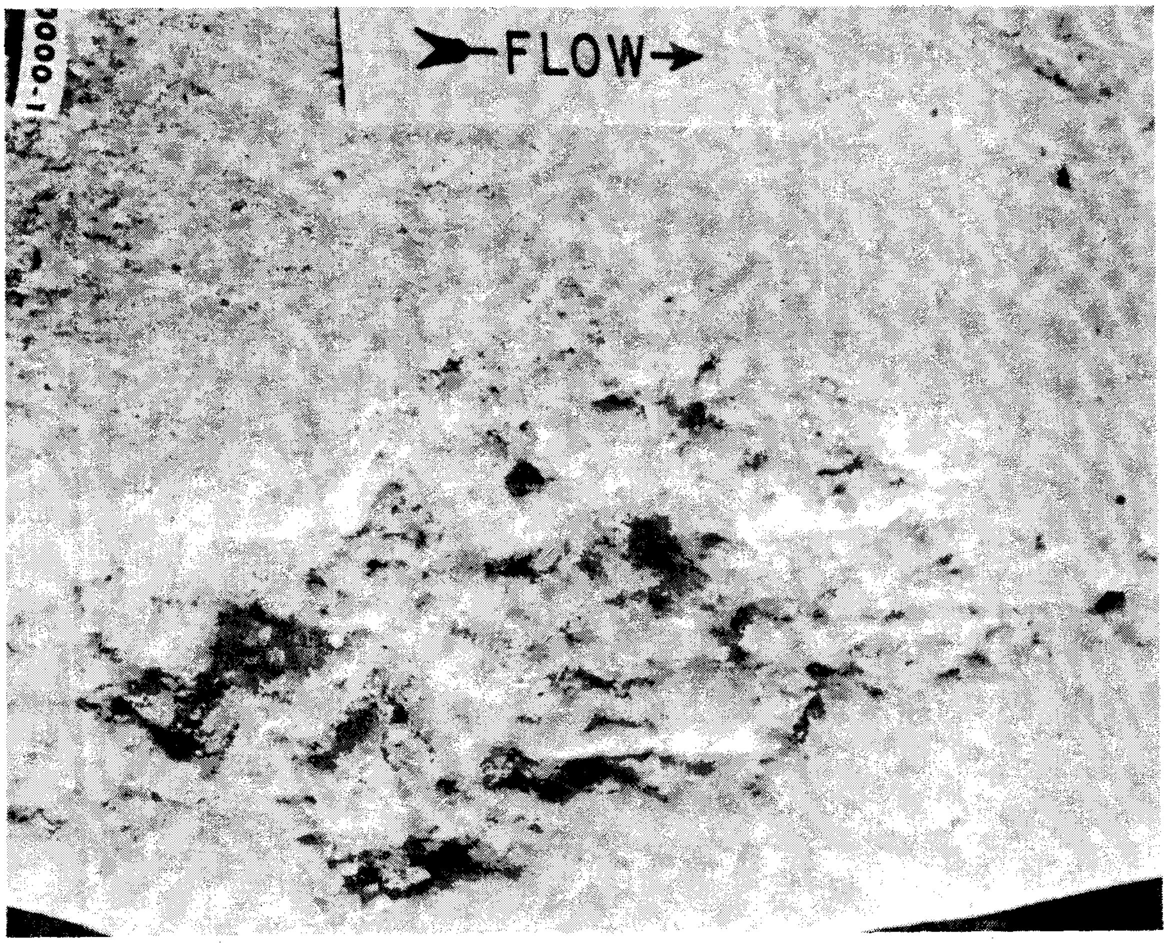


FIGURE 36--Another of the eroded areas is also shown full size. Depth of erosion here was 1/2 inch. The 1951 flood did not extend the size or depth of the eroded areas.

130, 000 acre-feet of water and at maximum water surface elevation it contains 480, 000 acre-feet. The drainage area is 3, 070 square miles. The service spillway is a morning-glory type which discharges into a horizontal tunnel under the dam as shown in figure 37. The emergency spillway on the left abutment is a saddle 1, 500 feet wide excavated to elevation 2297, which is 21 feet below the top of the dam. An expendable earth dike, 5 feet high, was constructed across the saddle to prevent unnecessary small flows through the emergency spillway. It was expected that a large flow would overtop the dike and wash it out.

An outlet works for release of irrigation water is located near the left riverbank. Since the outlet works has no physical connection with the spillway and was not in operation during the prototype tests, no further mention of it will be made in this report.

As at Heart Butte, the morning-glory spillway is designed to operate throughout the range of free discharge, the transition range between free and submerged discharge, and for submerged flows as great as 40 feet of water above the crest. Likewise the crest is uncontrolled but has six equally spaced piers placed radially in plan to reduce vortex action (figure 38). The spillway has an outside diameter of 33 feet, 8 inches. Its crest elevation is 2272.00. Its capacity is 5, 000 second-feet at normal reservoir elevation 2297.0, the elevation of the emergency spillway crest. At maximum reservoir elevation the spillway capacity is about 5, 700 second-feet.

The spillway crest profile merges into a short vertical shaft with a small deflector at its base, then into a 90° vertical bend. The bend leads into a horizontal tunnel 13 feet 6 inches in diameter and 520 feet long, which

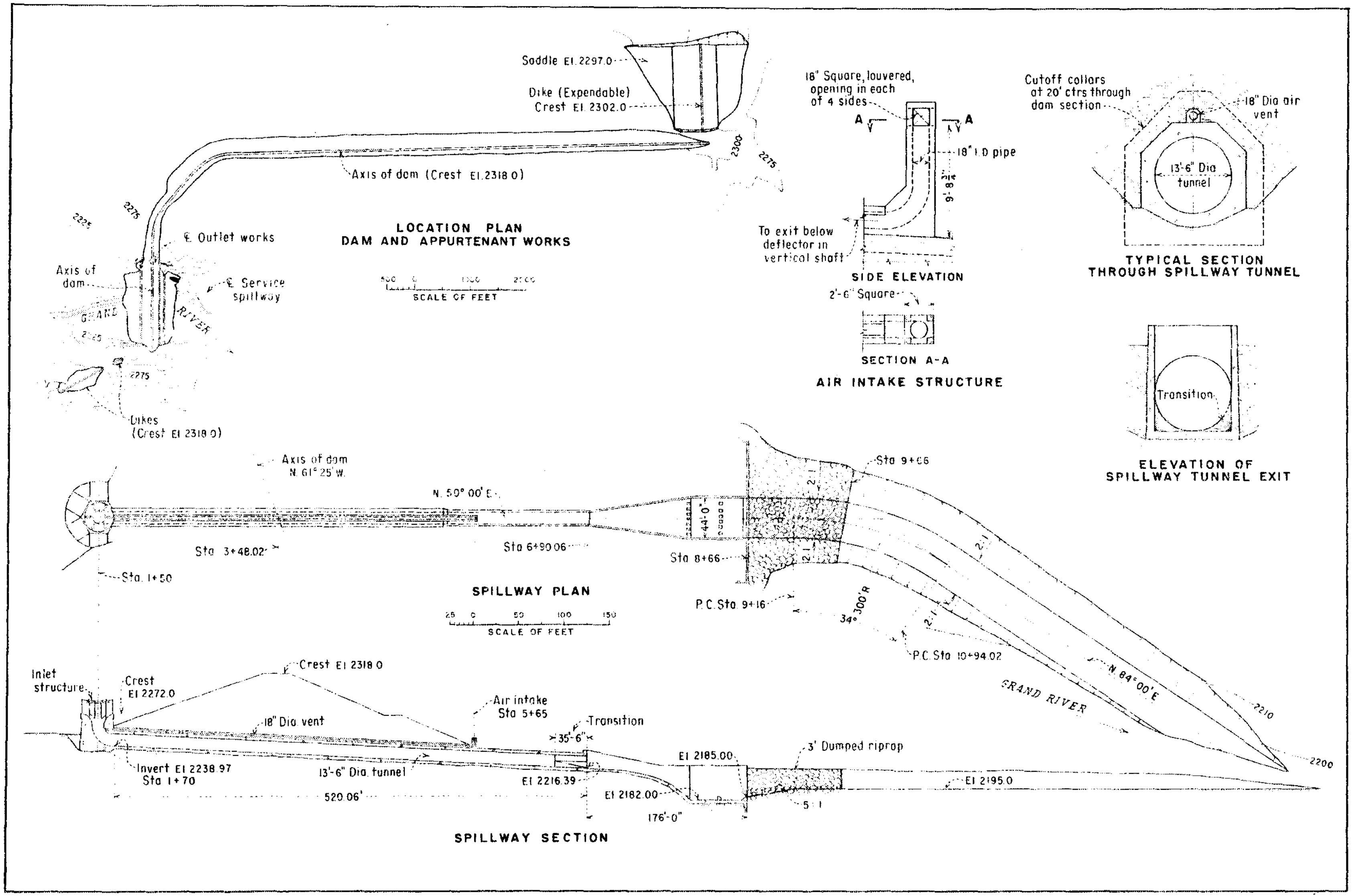


FIGURE 37--General plan and sections of the spillway at Shadehill Dam.

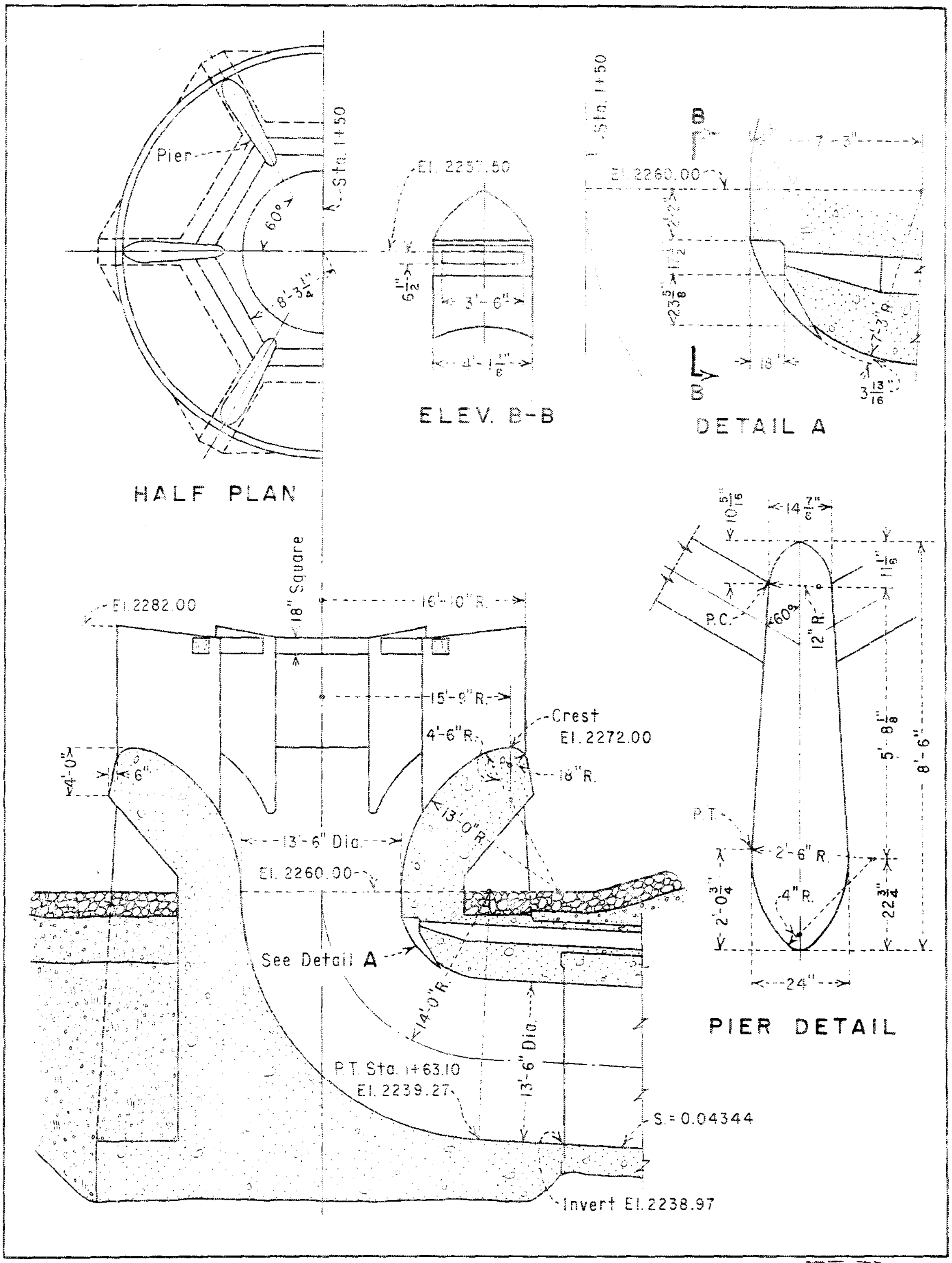


FIGURE 38--Major difference between the spillways at Heart Butte and Shadehill is that at Shadehill Dam the outlet works intake was not incorporated in the spillway structure, but was entirely separate.

terminates in a hydraulic-jump-type stilling basin. An excavated channel about 1,400 feet long conducts discharges from the stilling basin to the Grand River. Figures 37 and 38 show the general features of the spillway.

The stilling basin contains one row each of chute blocks and baffle piers and an end sill (figure 39). Spreader walls were not used in the basin. The maximum vertical fall from headwater to stilling basin floor is about 130 feet, which is the same as at Heart Butte.

The spillway approach, spillway, tunnel, stilling basin, and a portion of the downstream channel were tested and developed using a 1:20.73 scale model, see figure 40.

Summary of Model Tests

1. The model.--The Shadehill spillway model tests were made soon after the Heart Butte tests and since the structures were similar in many respects, the Heart Butte model was modified for the Shadehill tests.

The Shadehill studies were of the same general nature as the Heart Butte studies, and generally similar recommendations were made. The several important differences that do exist between the structures are discussed below.

2. Spillway air tests.--The air-entraining device used on the Heart Butte spillway was not used on the Shadehill spillway. On the Shadehill spillway, however, an 18-inch air vent was placed near the base of the deflector in the vertical shaft (figure 38), to supply air to the upstream end of the horizontal tunnel. The vent was located near the crown of the tunnel and supplied air to the space between the surface of the water and the tunnel crown; thus no air was introduced into the flowing water. The purpose of the vent was to reduce the possibility of low pressures occurring in the upstream end of the tunnel in the event that sufficient air did not flow upstream from the tunnel portal. This vent was not constructed in the model since it was not considered absolutely necessary for satisfactory operation. The decision to install the vent in the prototype was made after the model tests had been completed and it was installed as a precautionary measure.

3. Stilling basin tests.--Tests on the stilling basin were similar to those described for the Heart Butte basin except that it was found that spreader walls were not essential to good distribution in the basin. The lower angle of divergence in the basin was probably the factor that made the walls unnecessary. Model tests were made both with and without the walls in place and the tests showed that satisfactory distribution of the flow across the basin width was obtained without the dividing walls. Figure 41 shows the model in operation.

During the final tests to develop the most economical stilling basin, a question was raised as to whether the basin might be shortened an additional 14 feet. Tests were made on the shorter basin and its use in the prototype was considered. However, at that time it was decided that the longer basin would provide better protection. Therefore, most of the final data were taken on this basin (figure 39). After the model had been partially dismantled, the question was reconsidered and it was decided to use the shorter basin in the prototype. Thus, the graphic model-prototype comparisions of the stilling basin performance are not exact but are still valuable in indicating model-prototype conformance.

1952 Spring Flood

The cause of the flood in 1952 at Shadehill was similar to that at Heart Butte in 1950. Unseasonably warm weather following unusually heavy snow caused rapid melt and runoff. The rapid rise of the reservoir started on March 31 and continued for 10 days (figure 42). On April 10 the spillway crest was submerged 26 feet and 66 percent of the maximum water storage (84 percent of the storage below the dike crest) had been utilized. The rise during this period was a total of 46 feet to elevation 2298. The maximum inflow was 33,250 second-feet (figure 43). It was feared for a time that the expendable earth dike across the emergency spillway would be overtopped. After the snow disappeared from the ground, however, the inflow was sharply reduced and the reservoir reached a maximum height of about 1 foot above the base of the dike. The dike thus proved to be of value in preventing unnecessary discharge.

The morning-glory spillway went into operation on April 4. Maximum discharge

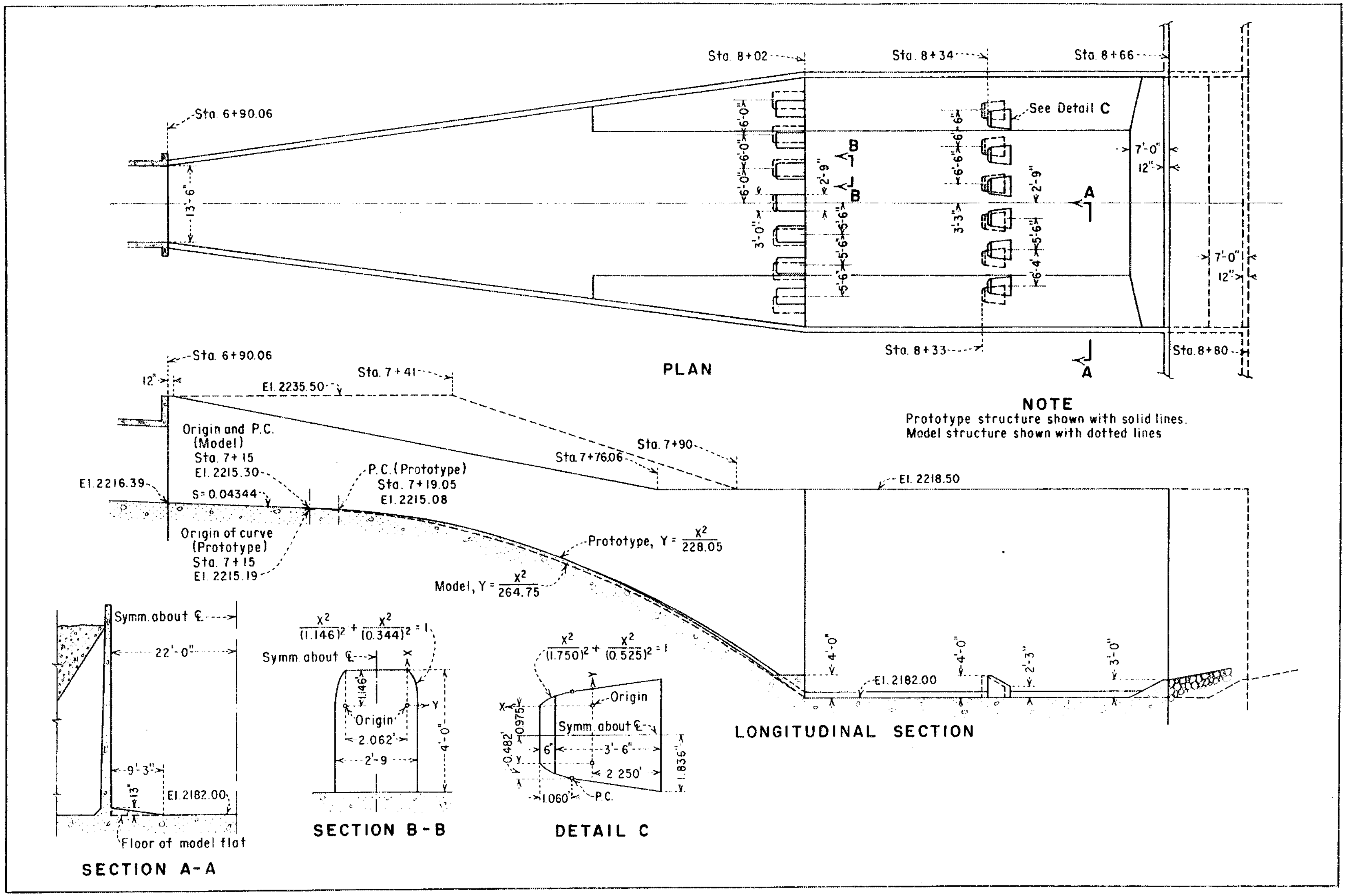


FIGURE 39--Stilling basin for Shadehill spillway, showing variation between model and prototype.

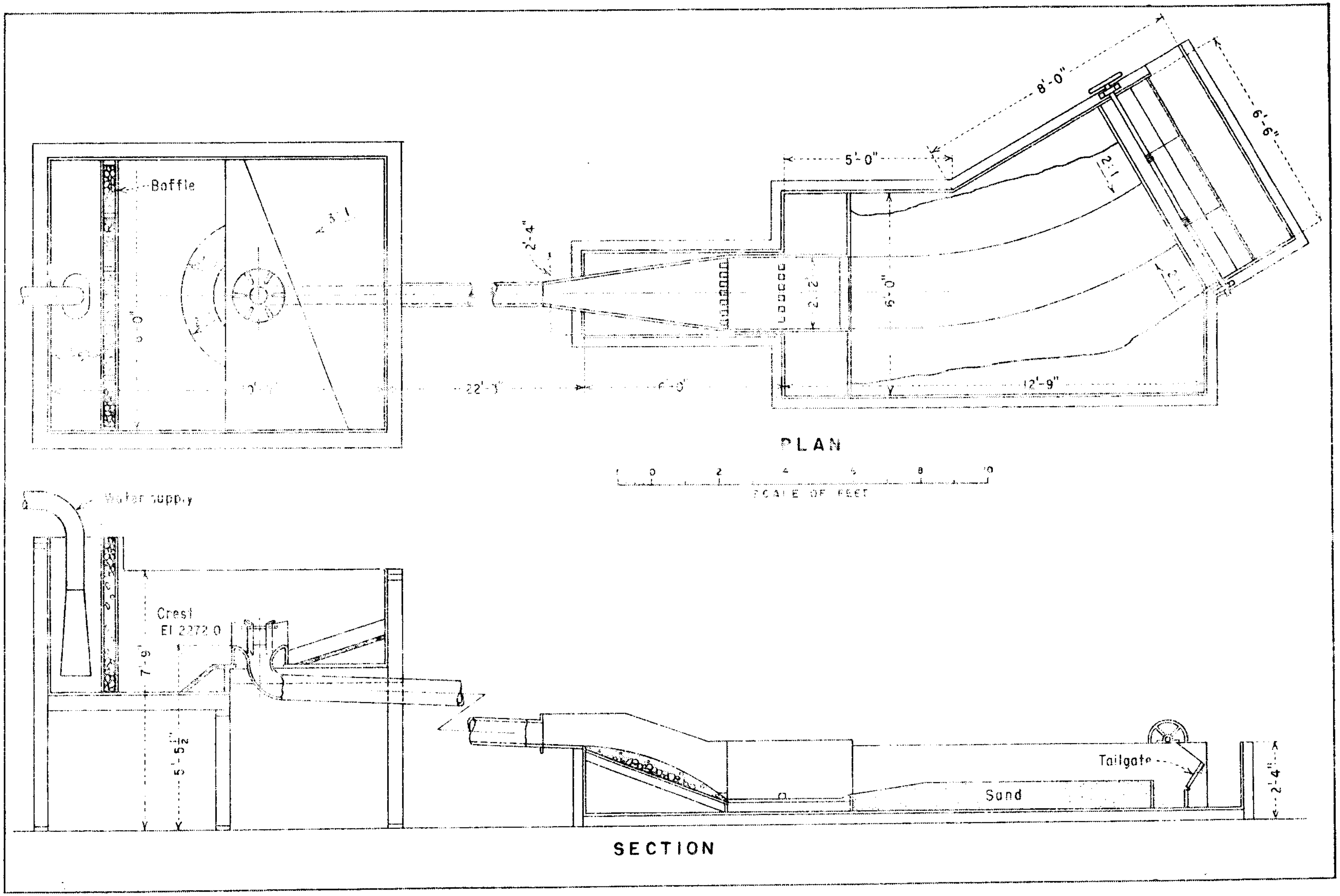
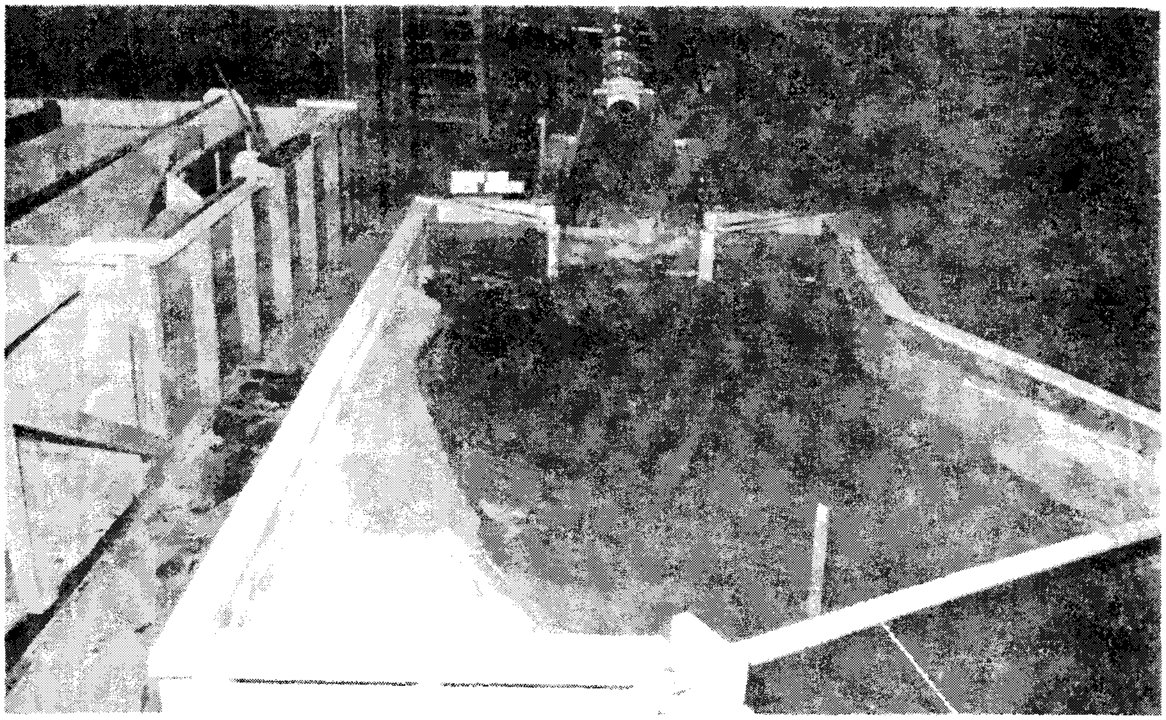
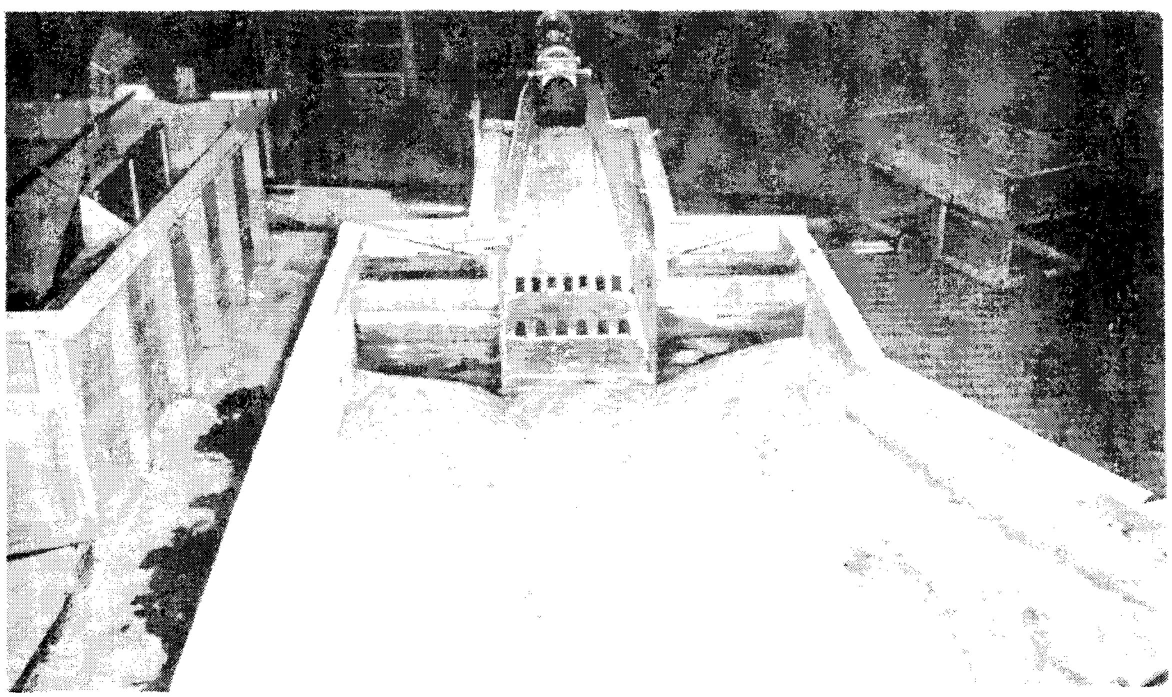


FIGURE 40--Shadehill Dam spillway model--1:20.732 scale.



A--Performance of the long basin shown in figure 39.



B--Erosion after 1-hour model test.

FIGURE 41--Test of the Shadehill spillway model at a discharge of 5,000 cfs and a tail water elevation of 2204.5.

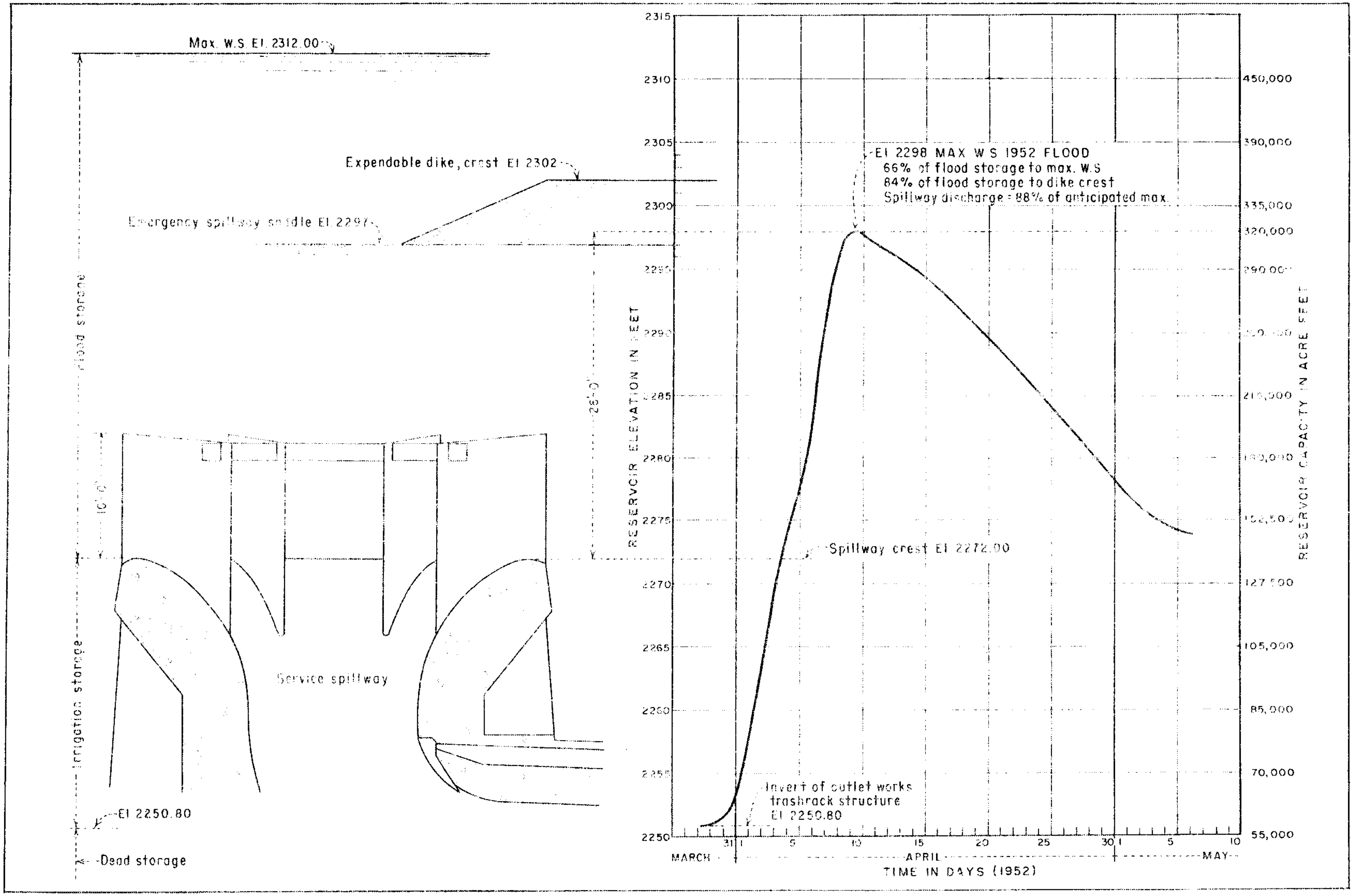


FIGURE 42—Reservoir elevations and hydraulic data of the 1952 flood at Shadehill Dam.

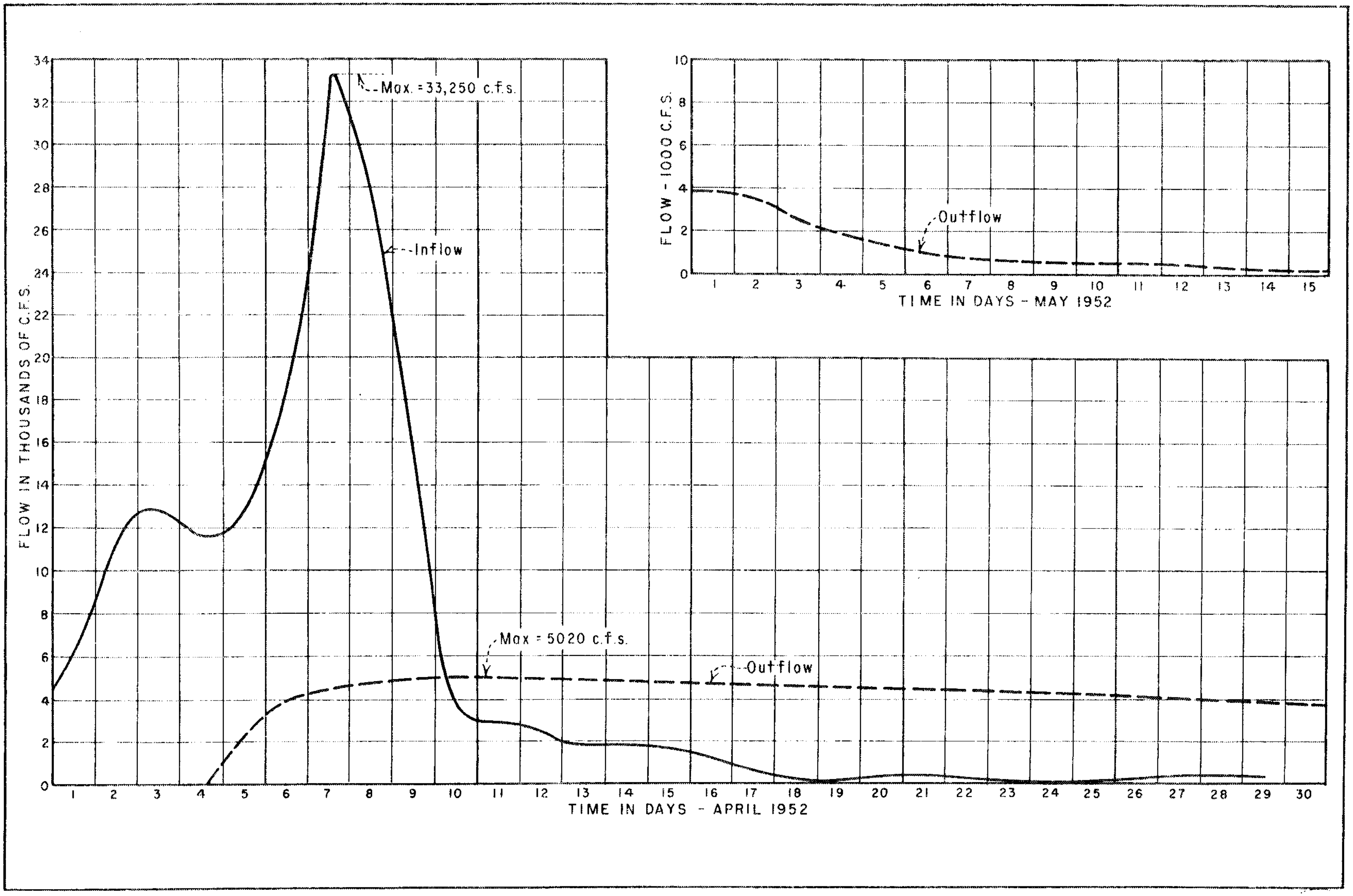


FIGURE 43--Hydrographs of the 1952 flood at Shadehill Dam.

was 5,020 second-feet on April 10 (figure 43). Extensive data were taken on the gradually decreasing discharge until May 7, when the flow was 800 second-feet. Spillway flow continued for several more weeks, however, before ceasing altogether. The spillway was thus in operation at appreciable discharges for more than a month, which provided a good opportunity to obtain data on the performance of the structure and ample time to evaluate the effectiveness of the stilling basin and riprap protection in the excavated channel.

The greatest spillway discharge during the flood represented 88 percent of maximum spillway capacity, while the highest reservoir elevation during the flood indicated that 66 percent of reservoir storage had been utilized. For these conditions the spillway crest was submerged 25.9 feet and the tops of the piers 15.9 feet. The maximum headwater to tail-water fall was 94.9 feet and the energy entering the stilling basin amounted to about 40,000 horsepower.

Model-Prototype Comparison Tests

Data on the performance of the Shadehill spillway were taken to compare model and prototype performance and to evaluate the performance of the entire prototype structure. Model-prototype comparison data were taken on spillway discharge, vortex action for submerged flow, head losses through the structure, water surface profiles in the stilling basin, and erosion below the stilling basin. Other data, taken to evaluate the performance of the structure, included wave heights, water surface profiles in the excavated channel, comparisons of the predicted and actual tailwater elevations, and the effectiveness of the riprap in preventing bank erosion.

1. Spillway capacity. --Headwater elevations were taken daily at 9 a.m., using the headwater gage located in the outlet works gate operating house. Reservoir elevations throughout the flood period are plotted in figure 42. An outflow hydrograph (figure 45) was prepared, using the reservoir elevations and the discharge-capacity curve obtained from the model. The discharge capacity curve is shown in figure 44. On the dates shown by the circles in figure 45 the United States Geological Survey made current meter measurements in the river downstream from the excavated channel to get an independent check on spillway discharges. The circled points show the river discharge obtained from the current meter traverses; the figures adjacent to the circles indicate the differences in percent between measured discharges and those predicted by the model tests.

The agreement between predicted and measured discharges is considered excellent. The average variation from the model calibration curve is only 2.7 percent for the 12 measurements made by the United States Geological Survey. In eight instances, the stream gaging measurements were less than the discharge predicted by the model. The average variation was 2.2 percent. In the four instances where the model predicted less water than was actually measured, the average variation was 3.6 percent. Since both plus and minus variations were small, and since the 12 measurements covered the discharge uniformly from 600 to 5,000 second-feet during both the rising and falling stages, it can be concluded that the model was capable in every respect of accurately predicting prototype discharges.

2. Spillway performance--submerged and free discharges. --As discussed for the Heart Butte tests, the model was designed to provide a minimum transition range, 0.2 foot, between free and submerged flow, or vice versa. The prototype tests indicated that this range was not exceeded by an appreciable amount; and, from evidence available, the transition in flow occurred without incident. As at Heart Butte, however, the flow transitions occurred without eyewitnesses. The rate of rise was so rapid during the filling cycle that field personnel could not be alerted in time to make the observation. During the draining cycle the flow changed from submerged to free during the night.

At 2:30 a.m., on May 2, however, the U.S.G.S. gaging station recorder, located in the downstream river channel, started to change slope, indicating a reduction in discharge not attributable to the uniformly decreasing head. When the slope change started, the headwater was at elevation 2276.96, which is the elevation at which the model indicated that free discharge would begin (figure 44).

Figure 46A is a photograph of the spill-

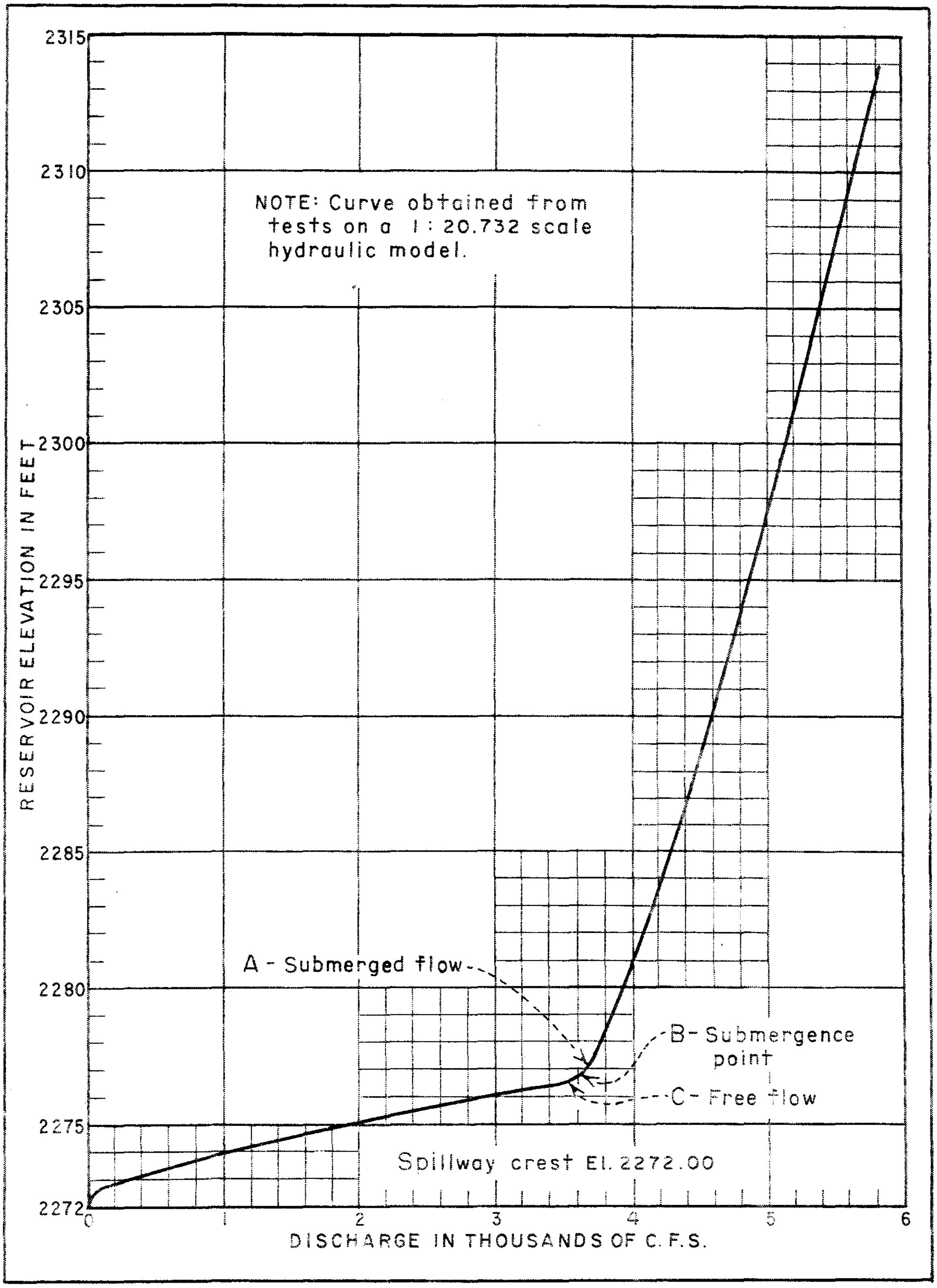


FIGURE 44--Discharge capacity curve of the service spillway at Shadehill Dam.

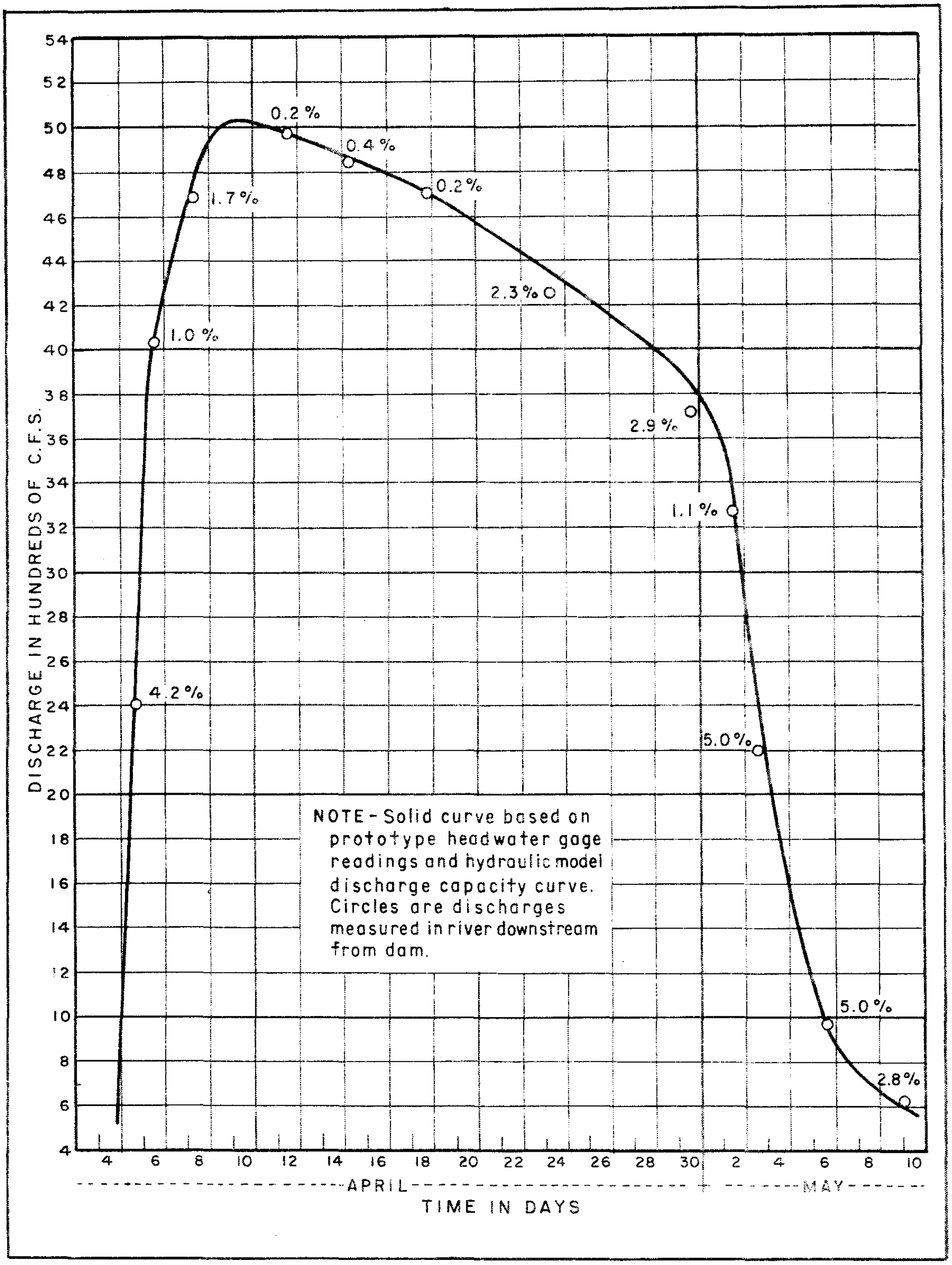
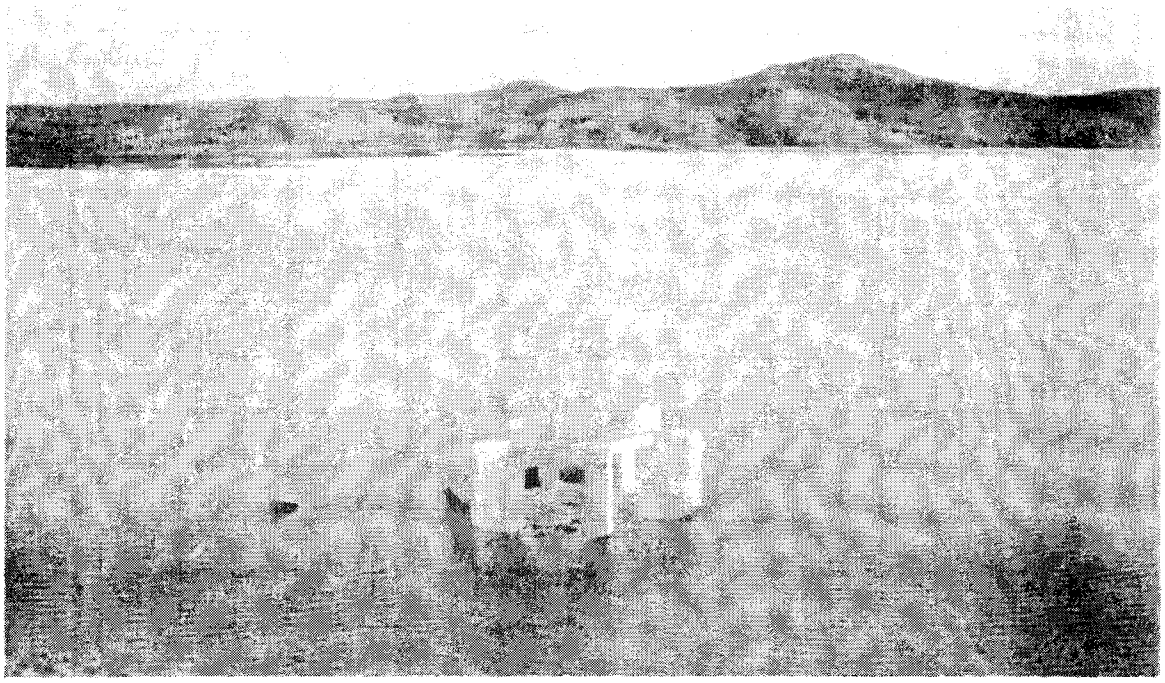
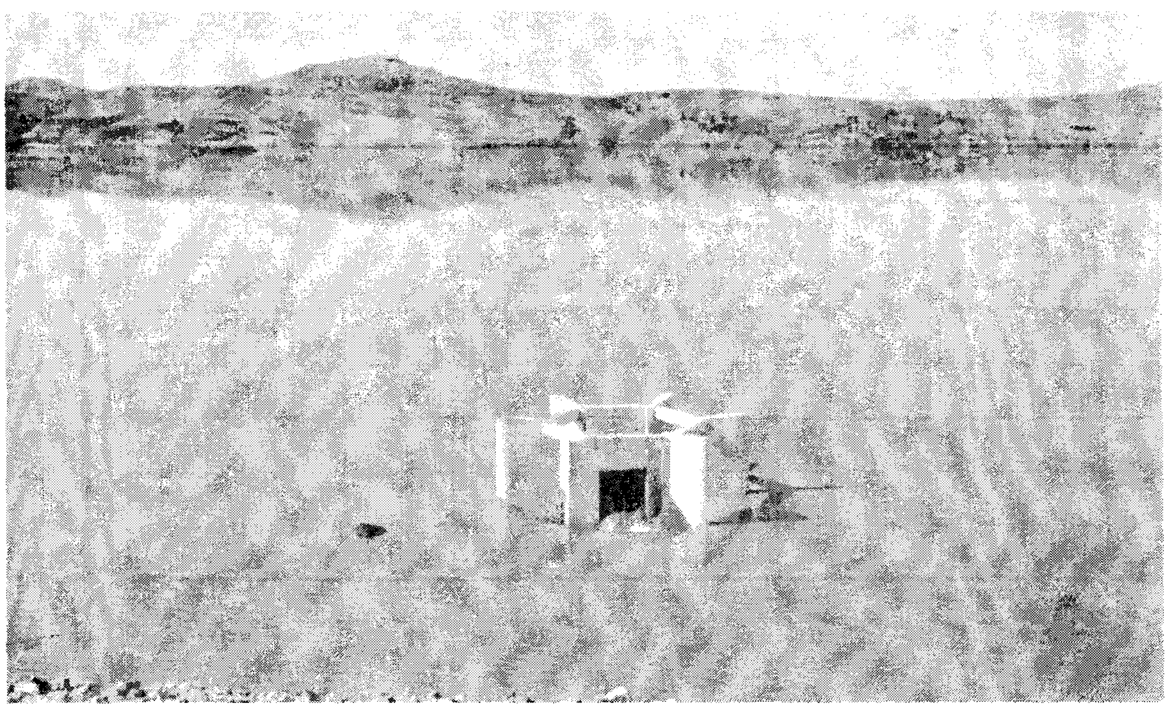


FIGURE 45—Comparison of the discharges of the model and prototype spillways at Shadehill Dam.



A--Submerged flow at headwater elevation 2277.2, about 0.3 foot above free discharge point. About 7:00 p. m., May 1, 1952.



B--Discharge changed to free flow during the night. At headwater elevation 2276.7, it is about 0.2 foot below free discharge point.

FIGURE 46--Transition from submerged to free flow at Shadehill Dam.

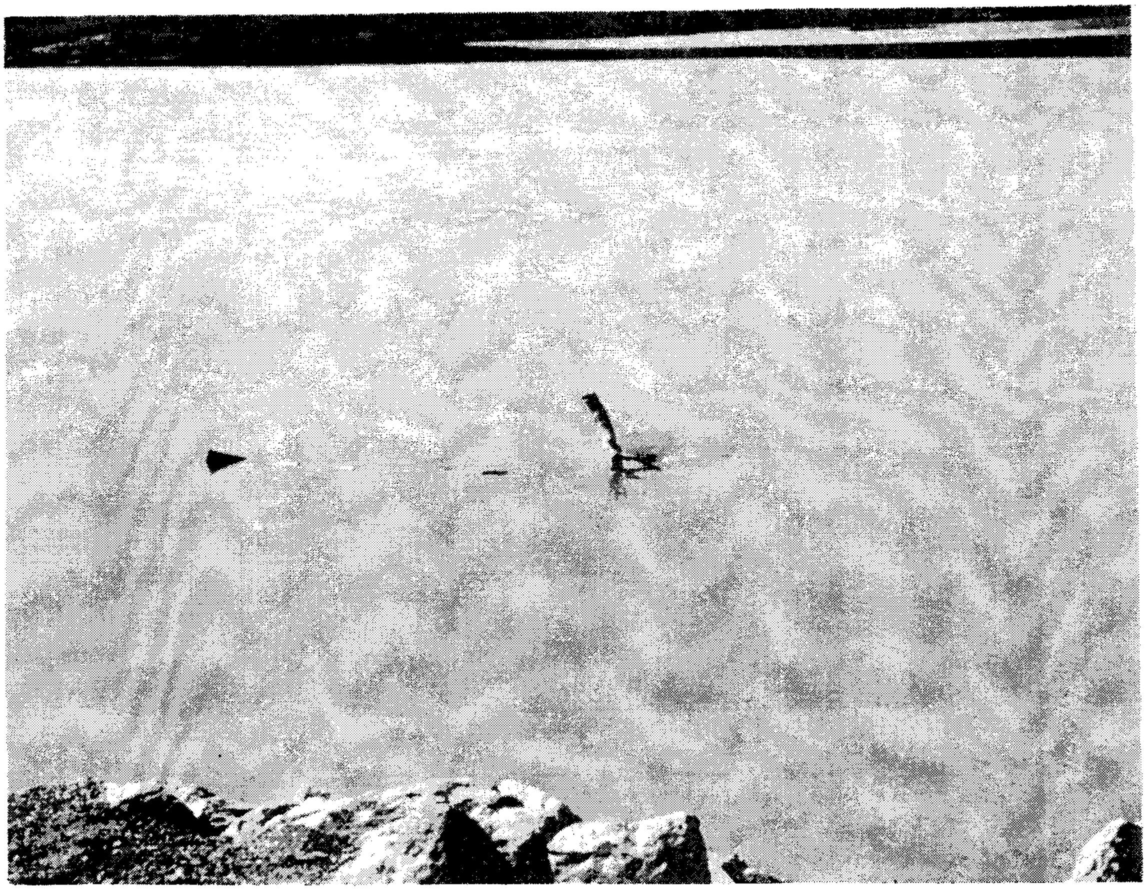


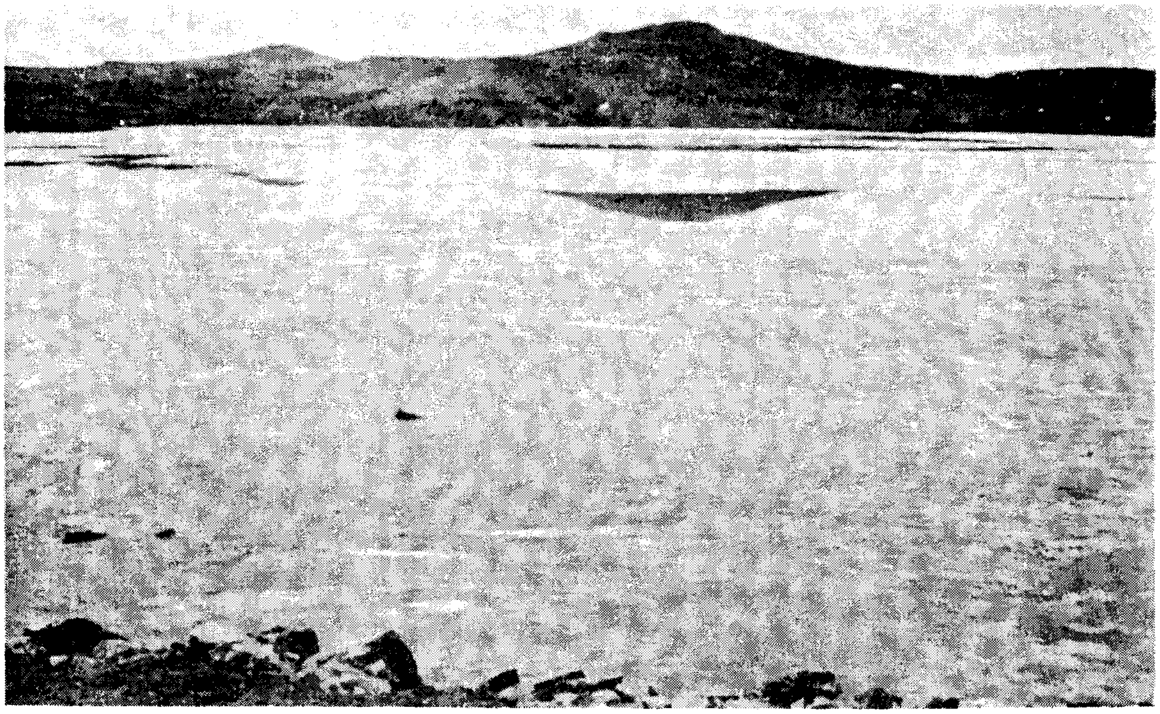
FIGURE 47--The vortex was continuous and did not shift laterally more than a few feet. There was no unusual noise. Tree trunk was 18 to 20 inches in diameter.

way taken just before dark on May 1. The headwater elevation was 2277.2 or about 0.3 foot above the free discharge point. The flow was smooth with no apparent pulsation, even though the "mushroom" had started to recede in the shaft. Early the next morning another photograph, figure 46B, was taken with headwater at elevation 2276.7, or about 0.2 foot below the free discharge point. The flow was free with no column of water visible in the shaft. The three elevations discussed above are plotted on figure 44 as Points A, B, and C, and indicate that the prototype performance was exactly as predicted by the model.

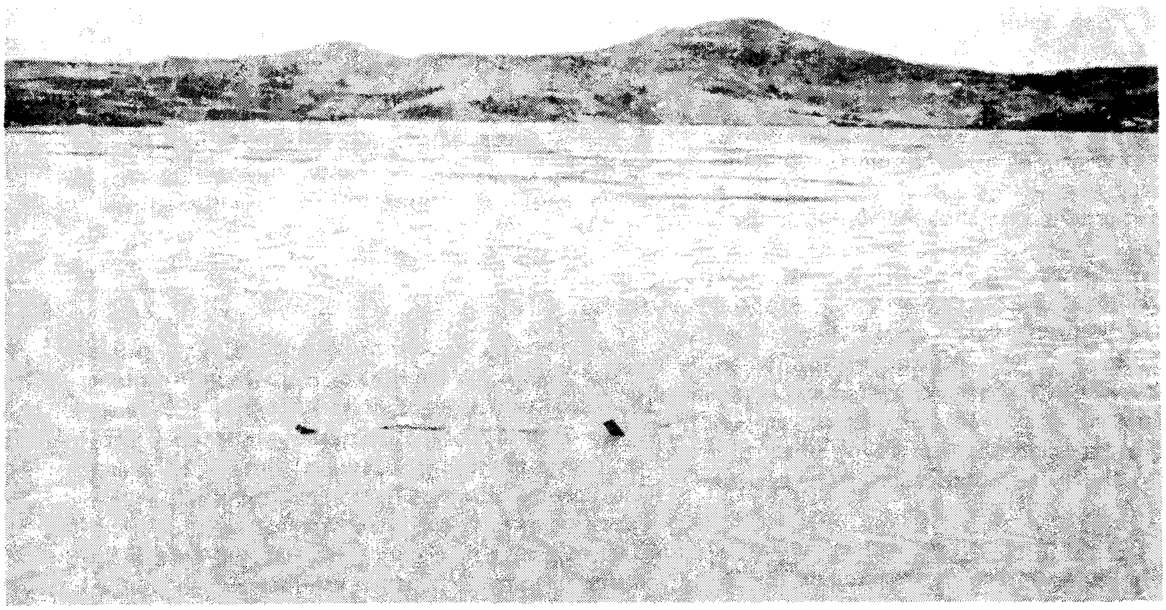
3. Vortex action and effect of ice.--Observations of the vortex formation during submerged conditions were of interest because they provided an evaluation of the effectiveness of the spillway piers in reducing vortex action. Model tests had shown that piers 14 feet high would provide the optimum reduction in vortex action. Taller piers had little additional effect in reducing vortex action while shorter piers were less effective in reducing vortex action. The appearance of the model in operation with and without spillway piers was practically identical to that illustrated in figure 4 for Heart Butte Dam.

The vortex observed at Shadehill Dam was somewhat larger (estimated to be twice the diameter) than expected for piers 14 feet high. For structural reasons, however, the prototype piers had been reduced to 10 feet. Undoubtedly this was the reason for the larger vortex observed. The vortex is shown in figure 47. Its size may be estimated by comparing it with the 20-inch diameter tree trunk lodged against the spillway. It is believed that the vortex would have been several times larger without piers.

On April 17, reservoir elevation 2293.2, the reservoir was covered with partially



A--Spillway area covered with decaying sheet ice about 4 inches thick. Morning glory is to right of tilted oil drum.



B--Sheet ice was broken up and discharged through vortex, keeping area around spillway free of ice.

FIGURE 48--Effect of ice on the vortex action of the Shadehill spillway.

decayed sheet ice about 4 to 6 inches thick (figure 48A). In the vicinity of the spillway the ice appeared to be in continual vertical motion. Ice particles seemed to be rising and falling and to be causing a tinkling sound which could be heard plainly on top of the dam. It is believed that the partially decayed ice was being subjected to compressive forces by the converging flow, causing the ice to shatter and splinter. Other action over the spillway was so mild, however, that it was difficult to find the spillway location from the appearance of the reservoir surface. On April 18 the reservoir was 0.2 foot lower and a fairly large area near the spillway was free of ice. A vortex was evident at all times, see figure 48B. The vortex was stable and did not change its location laterally more than a few feet during the entire day. The model vortex was less stable in this respect, being in lateral motion almost continuously. This lateral movement of the vortex in the model may have been caused by the greater amount of turbulence in the flow approaching the model spillway.

Occasionally an ice sheet drifted over the prototype vortex opening, causing the vortex action to stop momentarily. But after a few seconds, the ice sheet would disintegrate and fairly large pieces of ice would be carried down by the vortex. The vortex resumed its normal action when the entire sheet had been broken up and discharged through the spillway.

The strength of the vortex was demonstrated by the ease with which the large sheets of ice were broken and discharged through the tunnel. This strength was also manifested when it was found that several of the 50-gallon drums used as floats to hold the safety boom in place around the spillway had torn loose from their moorings and had been passed

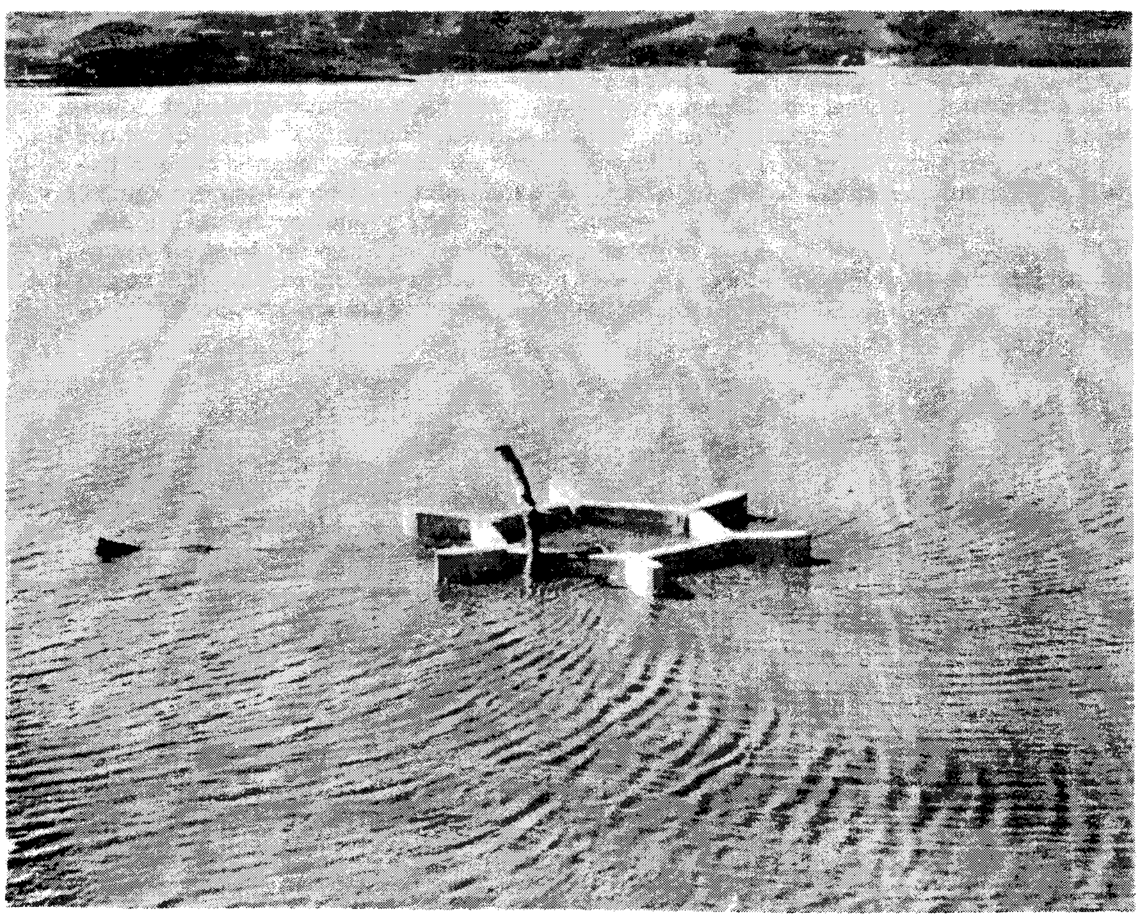


FIGURE 49--At headwater elevations above this point, elevation 2284.9, vortex action was continuous. The vortex is partially hidden by a pier strut.

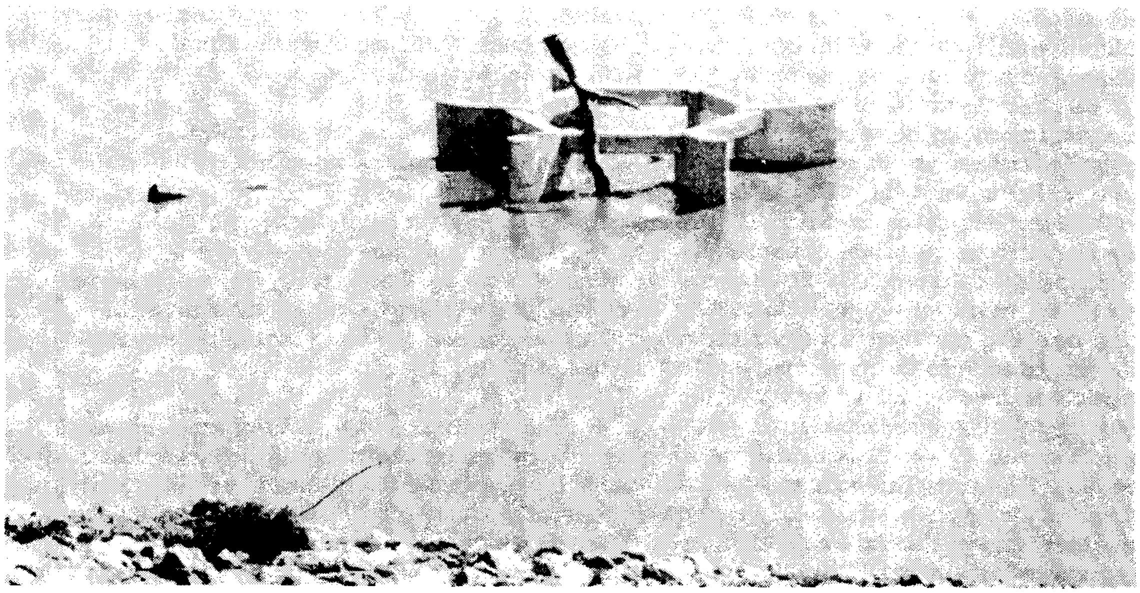


FIGURE 50--At headwater elevation 2283.6, vortex action was sporadic, with surface waves a few inches high sufficient to stop the action.

through the vortex and discharged through the tunnel. Two of the drums had caught in the branches of two partially submerged trees in the excavated channel downstream from the stilling basin. Other drums could not be accounted for and it was presumed that these too had passed through the spillway and on downstream.

Despite its size and demonstrated force, the vortex caused no apparent ill effects on the structure or its operation. An audible gurgling sound accompanied the vortex action but this sound was easily drowned by ordinary conversation, wind noises, or other normal sounds. The force of the vortex did not extend appreciably beyond the visible limits of the swirling water, although a small boat passing close to the vortex would probably be endangered. Evidently if the spillway piers had been shorter or eliminated altogether, the vortex and consequently its

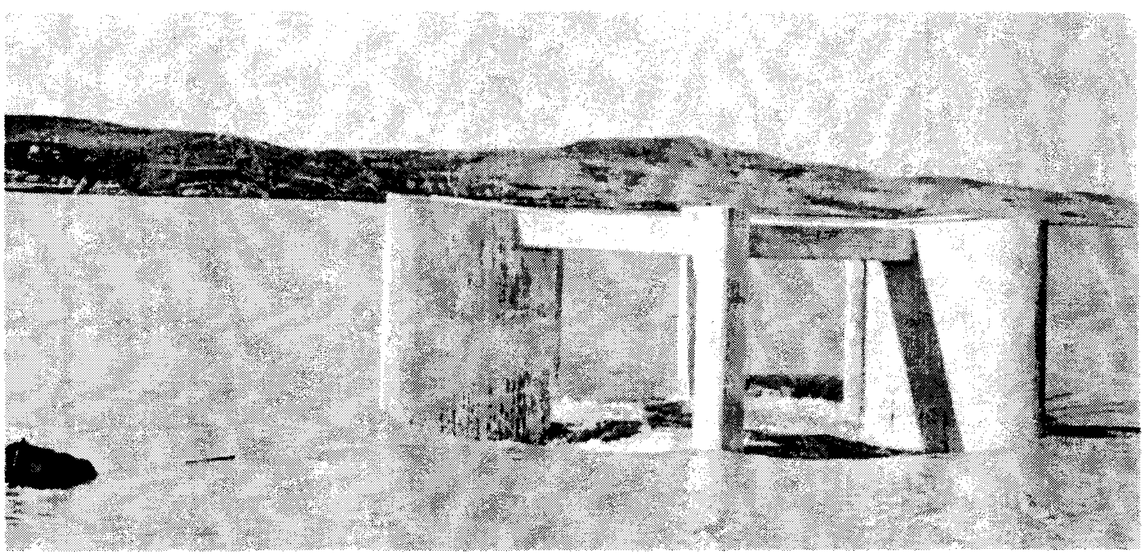


FIGURE 51--No vortex was observed when the water surface in the shaft was below reservoir level. Elevation here was 2277.8, about 1 foot above submergence.

effects would have been larger.

When ice was not covering the spillway area, vortex action occurred continuously from reservoir elevation 2298 to about elevation 2285. Between elevations 2285 and 2279 vortex action was sporadic and was affected by the turbulence in the spillway opening and by waves on the reservoir surface. No vortex action could be detected below elevation 2279. Figures 49, 50, and 51 show spillway performance and vortex action at three different headwater elevations.

4. Vibration.--Throughout the flow range no vibration could be felt in the structure. While the opportunity to get close to the spillway tunnel was less than at Heart

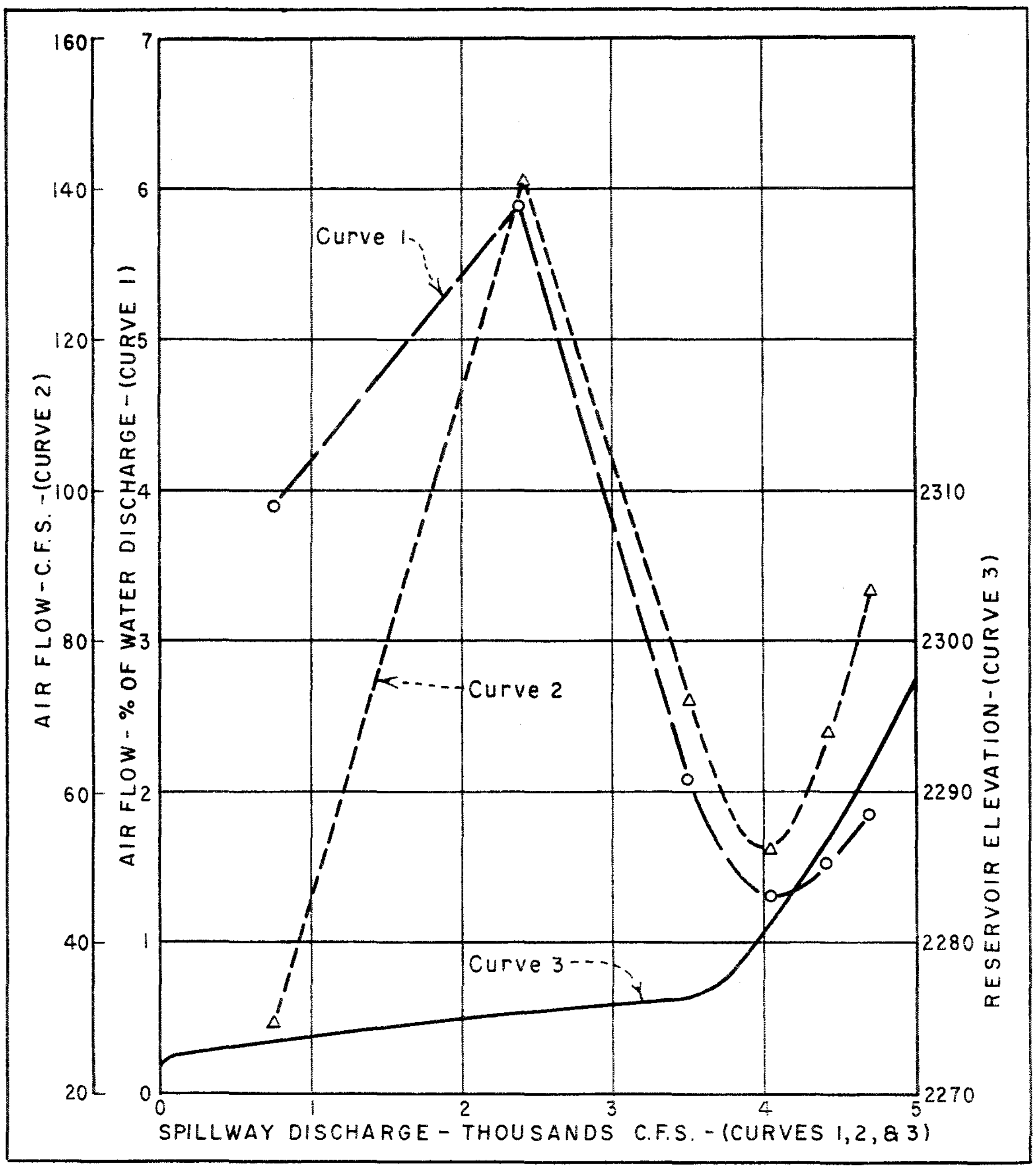


FIGURE 52--Air demand and spillway discharge curves for Shadehill Dam.

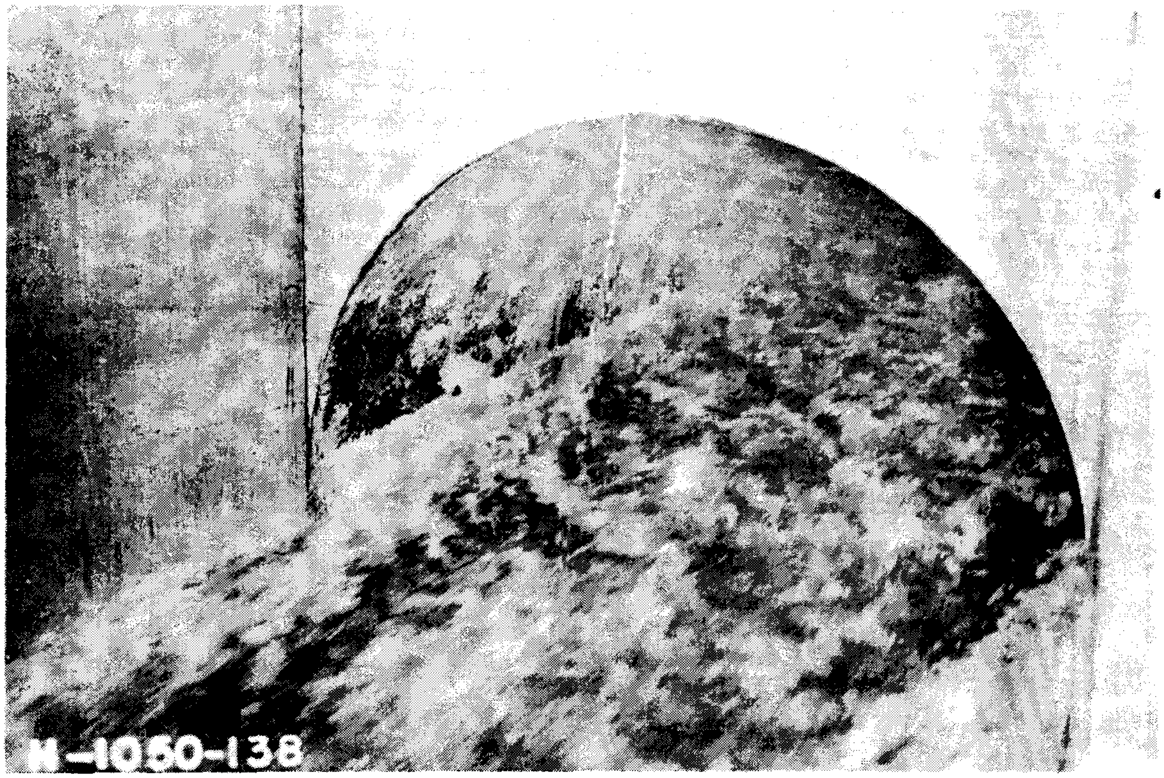


FIGURE 53--Trickle of water dropping straight down into flow at tunnel portal shows that air flow through the tunnel was not sufficient to deflect the trickle. Straw dropped into the flow showed only slight air movement.

Butte, vibration was not perceptible in the parts that were accessible. Despite the pounding of the water in the stilling basin, no vibration of the stilling basin walls could be felt.

5. Debris.--The reservoir was remarkably free of floating debris. The dead tree visible in the photographs of spillway operation was the only large piece in evidence throughout the flood. This tree became lodged against the outside periphery of the spillway and piers but had no noticeable effect on spillway performance. On the morning of May 1, 1952, the falling reservoir exposed a large portion of the tree, and it apparently toppled over and was passed through the spillway. Observers watching the tree trunk float downstream after passing through the tunnel estimated its over-all length as about 30 feet.

6. Tunnel air demand.--The quantity of air entering the 18-inch-diameter vent located at the base of the deflector in the vertical shaft, figure 38, was measured using an anemometer held in the 18-inch-diameter intake pipe located at ground level just upstream from the stilling basin (figures 37 and 57A). The lineal feet of air flow was measured for six different discharges using an anemometer well braced in the pipe. Each measurement was sufficiently long to establish an average value. The quantity of flow was then computed and plotted against water discharge (Curve 2 of figure 52). The maximum air quantity measured was 141 second-feet and occurred for a water discharge of 2,400 second-feet. The corresponding velocity in the air pipe was about 80 feet per second. Since air vents will work satisfactorily with velocities up to about 300 feet per second, this air demand is regarded as moderate. The spillway rating curve is included as Curve 3 in figure 52 to show the relationship between air and water discharges. Curve 1 shows air flow as a percent of the water flow. Together, Curves 1 and 2 show that the quantity of air increases with increasing discharge until the water flow reaches about 2,400 second-feet, then decreases rapidly until after

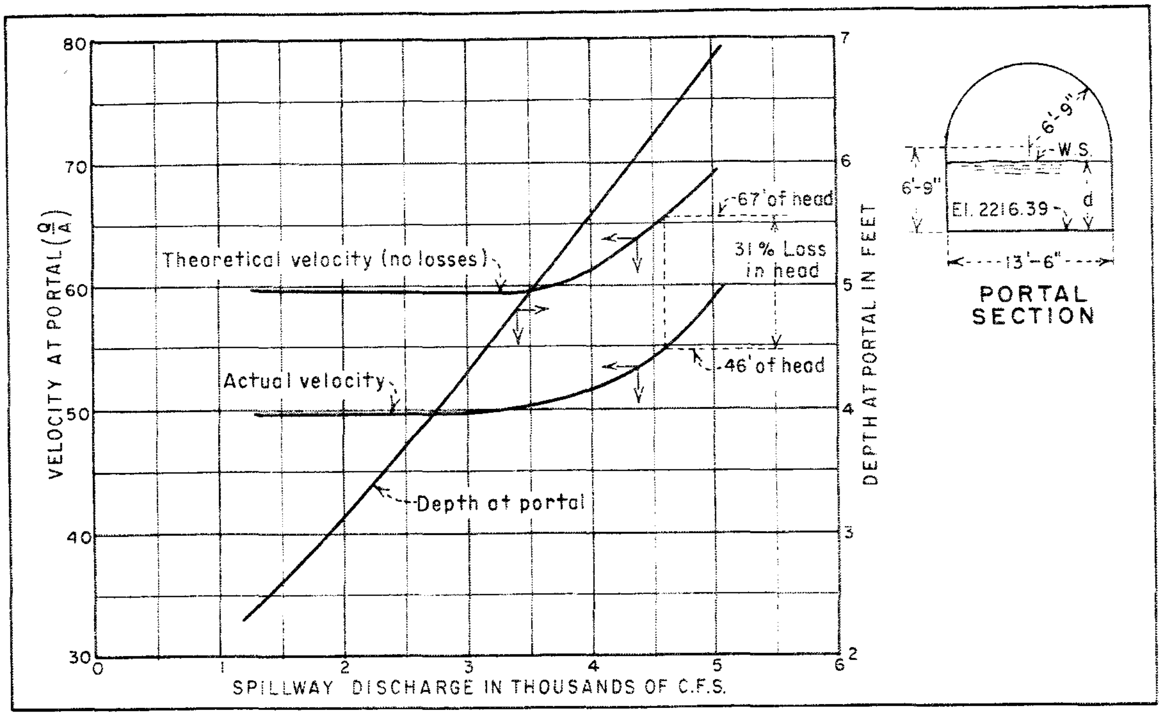


FIGURE 54--Overall head loss through Shadehill service spillway structure.

the spillway submerges, or until the water flow is about 4,000 second-feet. Air flow then increases with water flow.

Attempts to rationalize the quantity of air flow over the measured range have met with little success. Air can and probably does enter the tunnel through the spillway shaft until the spillway submerges. It is possible for air to enter from the tunnel portal at all times, but observations and tests made at the tunnel portal showed that air entered the tunnel portal only occasionally during flows of the order of 5,000 second-feet. Handfuls of straw dropped down the face of the head wall at the tunnel portal usually fell straight down to within a few inches of the water surface, then were deflected downstream a foot or two before striking the water surface. Water dripping from a drain pipe fell straight down at all times (figure 53), and was not deflected by the air currents. Part of the straw was occasionally drawn into the tunnel at the tunnel crown, indicating an occasional slight movement of air along the crown in an upstream direction. At the same time there was a downstream motion of air close to the water surface.

7. Spillway head loss.--Flow emerging from the tunnel had a relatively smooth, level surface and was not excessively insufflated, see figure 53. This made it possible to obtain reasonably accurate depth measurements at the portal. The depth was determined for eight discharges of from 1,300 to 4,700 second-feet. When plotted against discharge the depths made a smooth curve (figure 54). The average velocity at the portal was then computed from the discharge and flow area, and is plotted against discharge in figure 54. Theoretical velocities at the portal were computed assuming no head losses through the structure; these are also shown plotted against discharge in figure 54. The resulting velocity curves are parallel, indicating a difference between theoretical and actual velocity of about 10 feet per second. At 4,700 second-feet this amounts to a head loss of 21 feet, or 31 percent. At 1,500 second-feet the head loss was 17 feet, also 31 percent. Losses were also computed from data obtained during the model tests. Using the model depth of 6.3 feet for 5,000 second-feet, and reservoir elevation 2297.3, the head loss was computed to be about 20 feet, an over-all loss of 27 percent. This compares favorably with

the 31 percent head loss measured in the prototype. These figures indicate that the model, built to a geometric scale of 1:23.5 and constructed entirely of glassy-smooth transparent plastic, truly represented the prototype as regards velocities, depths, and energy losses.

8. Stilling basin performance.--During the hydraulic model tests, a basin was developed and recommended for field construction that was believed to be the minimum basin possible for good performance at maximum discharge. To reduce the cost of the basin it was proposed, however, that a basin 14 feet shorter than the recommended be tested. This shorter basin was not as effective as the longer basin in providing smooth flow in the channel, but it was deemed that the shorter basin would provide adequate energy dissipation to prevent channel bottom erosion even though the waves and surface disturbances would be more evident. The shorter basin was adopted after the model tests had been completed. During preparation of the construction drawings it became necessary to modify the upper end of the basin somewhat and consequently the basin constructed in the field was not identical to the one model-tested. Figure 39 shows model and prototype basins and indicates the differences in dimensions.

Figures 55 and 56 show water surface profiles along the prototype stilling basin wall for eight discharges between 1,300 and 4,700 second-feet, figure 55 also shows the water surface profile along the center line of the model for a discharge of 5,000 second-feet. A comparison of the profiles indicates that the differences in model and prototype basins caused little differences in the expected profile, and that for all practical purposes the prototype stilling basin performance was satisfactorily predicted by the model tests. Figure 57A is a general view of the stilling basin, downstream channel, and the Grand River. Flow throughout the expanding transition of the stilling basin was well distributed laterally and appeared to be of the same general pattern as that observed in the model, see figure 57B. The fins along each side wall were present in both model and prototype.

As at Heart Butte a considerable amount of water sprayed into the air, mostly at the point where the flow plunged beneath the tail-water. When there was no wind the spray fell back into the basin; at other times the spray was carried to areas adjacent to the basin. Under the worst conditions, the backfill along the basin training walls, downstream from the toe of the jump, became muddy and difficult to walk on. However, no washing of the backfill or other damage was evident, see figure 57B.

The close grouping of the profiles of figures 55 and 56 is probably caused to some extent by the extremely turbulent water surface in the basin. Part of the hydraulic jump was shrouded in mist and the surface rose and fell as much as 10 feet at the toe of the jump, making it difficult to obtain an average profile. At the end of the jump the surface was more quiet, but a large boil was evident over the end sill with surface fluctuations of several feet. The boil was more prominent than expected and later investigations indicated that the river tail-water, which was lower than that used to design and test the model basins, was a contributing factor.

9. Tail-water elevations.--During spillway operation it was readily apparent that the tail-water elevation in the downstream portion of the excavated channel and in the river was lower than expected from the tail-water curve computed prior to construction of the dam. This was not so apparent immediately downstream from the basin because the turbulent boils extended into the downstream channel (figure 58A). Profiles taken throughout the length of the excavated channel for seven different discharges indicate that the tail-water was at least 1 foot too low at the end of the basin measured at a point 5 feet to the left of the left training wall on the downstream face of the 90° wing wall, and 2 feet too low 20 feet to the left. The exact deviation could not be determined at these points because of the boil, and its effects downstream and laterally at the end of the basin. At a point 110 feet downstream from the basin, the profile indicated that the tail-water was 1.2 feet below the expected elevation for 5,000 second-feet. Where the excavated channel emptied into the river the tail-water was about 3 feet too low, (figure 59). Since the computed tail-water curve was for the river channel and not the excavated channel, it was expected during model testing that tail-water elevations at the basin

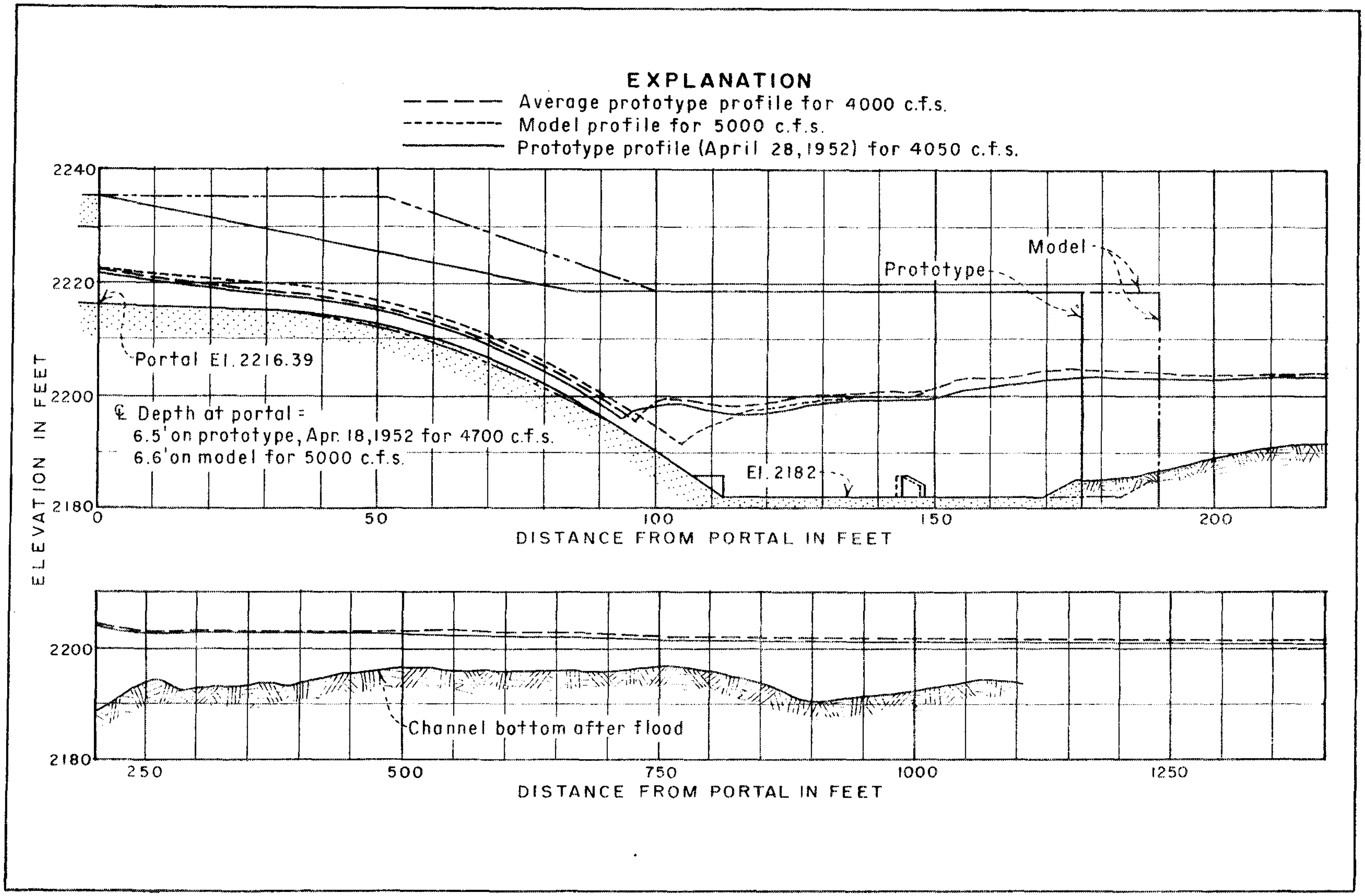


FIGURE 55--Comparison of water surface profiles of model and prototype spillways.

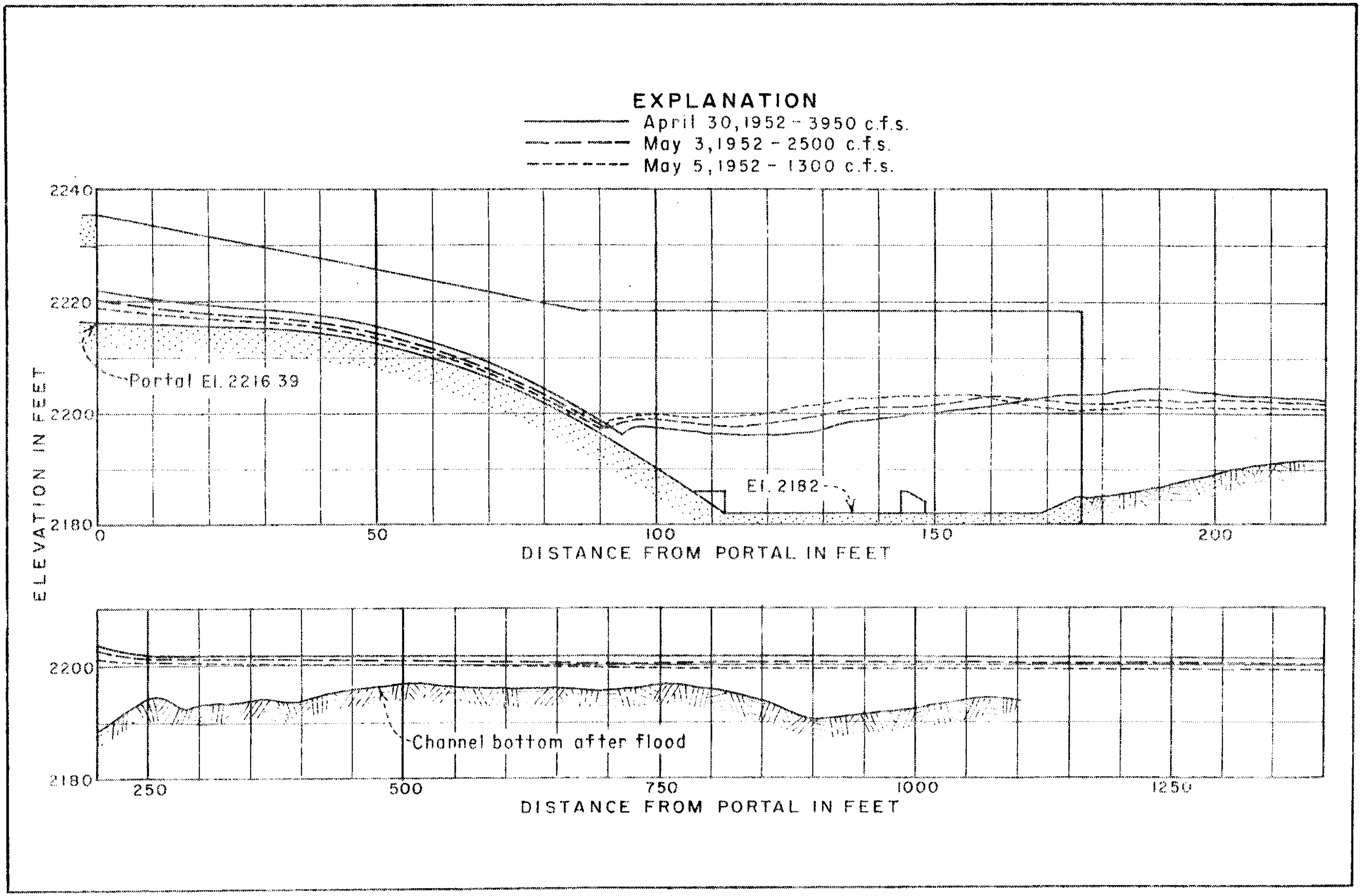
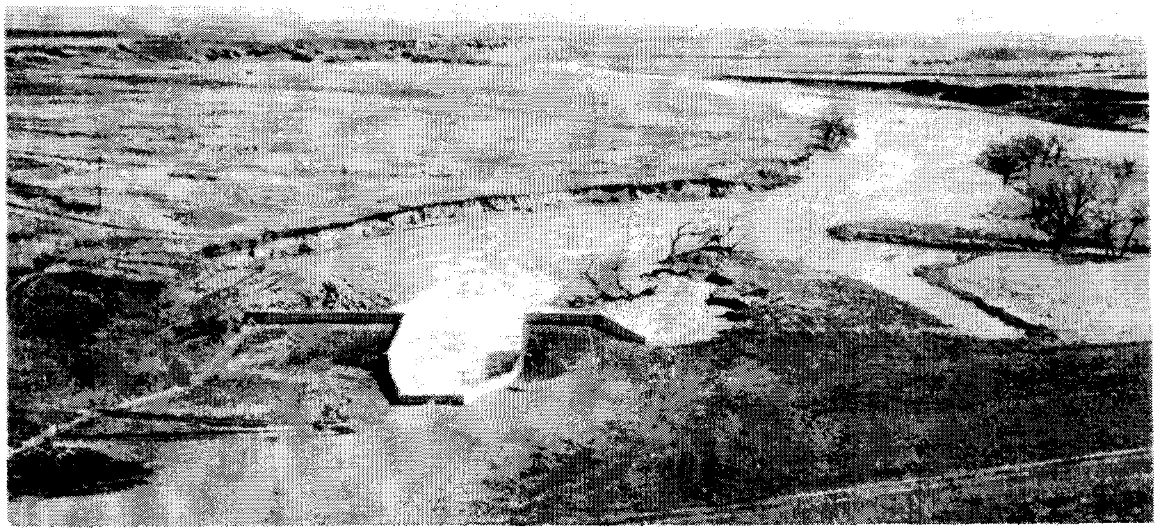
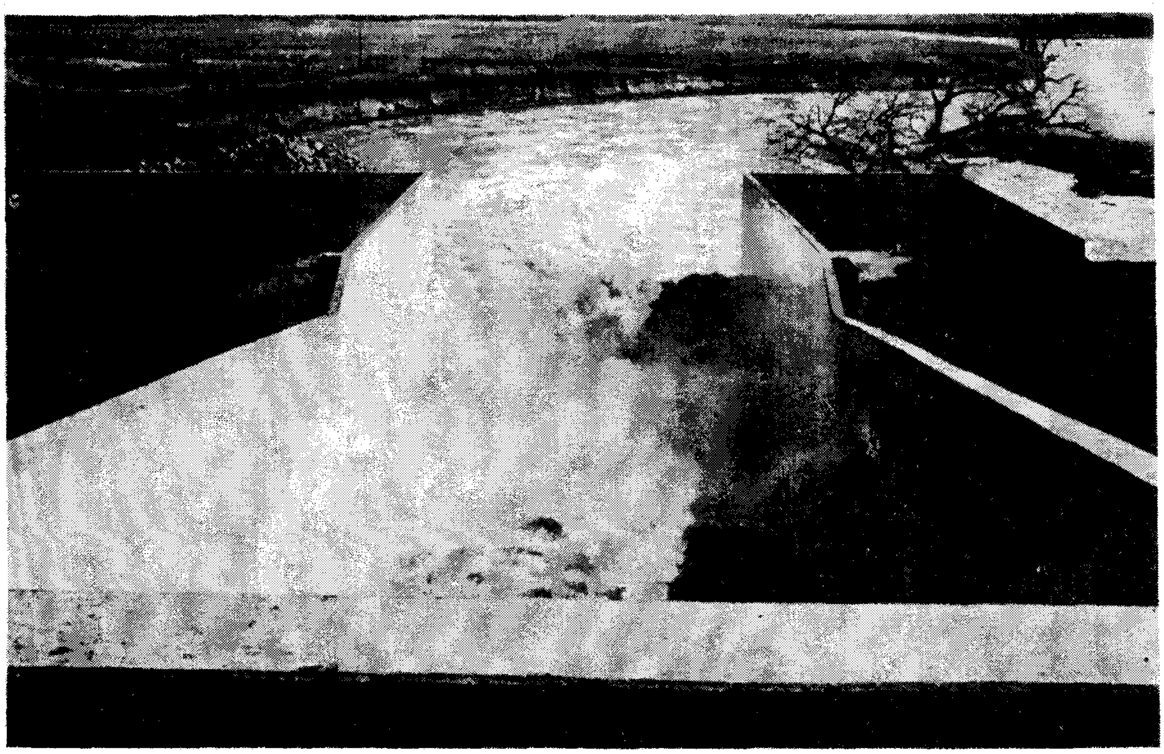


FIGURE 56--Water surface profiles of Shadehill spillway, 1,300 to 3,950 cfs.

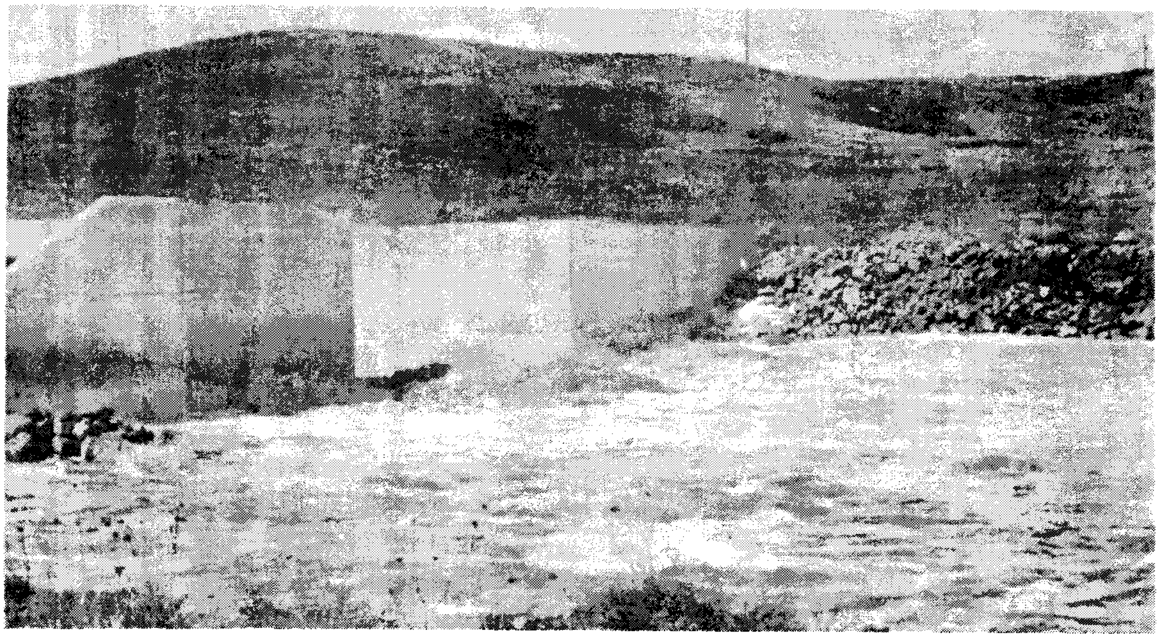


A--Stilling basin, excavated channel, and the Grand River on April 24. Air intake structure in the foreground.

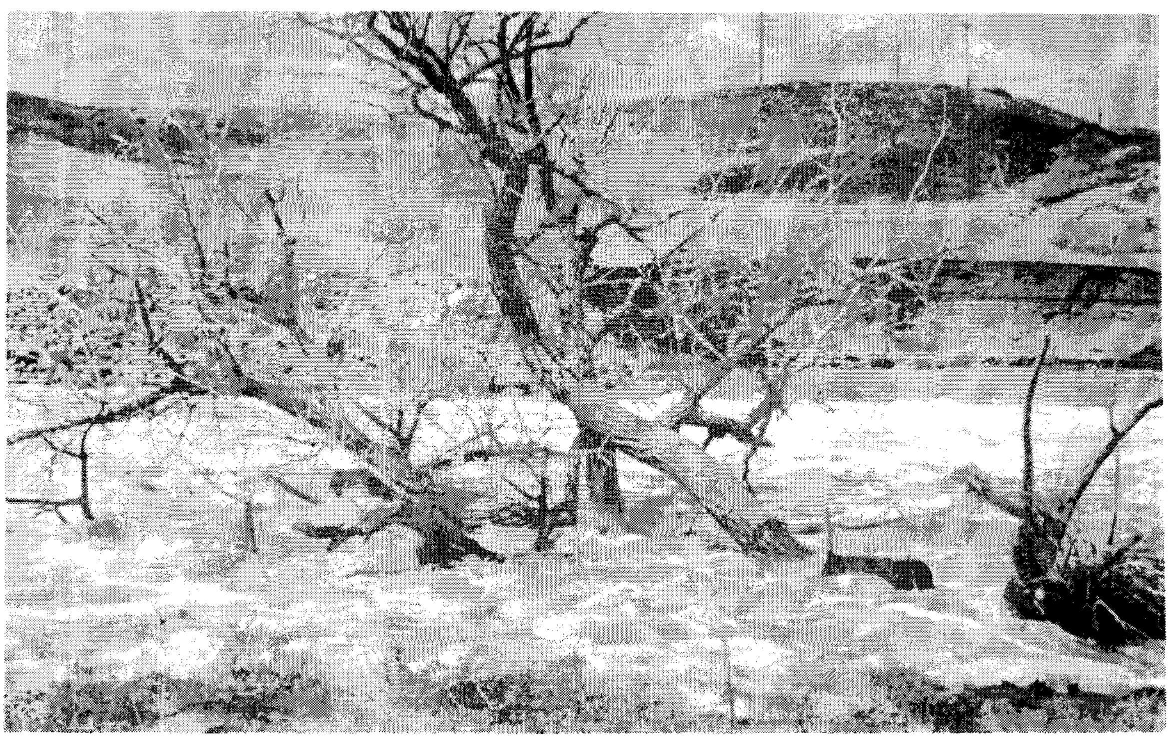


B--Stilling basin performance showing turbulence at the toe of the hydraulic jump and fins along walls of the transition. Spray from the jump caused wet areas adjacent to the walls.

FIGURE 57--Shadehill spillway stilling basin discharging 4,400 cfs.



A--Boil at end of stilling basin extended into excavated channel, making exact determination of tail water elevation difficult.



B--Erosion of channel banks isolated trees and widened the channel. Far bank was eroded by eddy cutting behind riprap. Oil drums caught in trees were part of safety boom that had passed through spillway.

FIGURE 58--Shadehill spillway stilling basin discharging 4,600 cfs.

would be higher than the values shown on the computed curve. Because these tail-water elevations were lower, rather than higher than those anticipated, the operation of the stilling basin was adversely affected to a greater extent than the figures above indicate.

This low tail-water caused the boil at the end of the basin to be higher, the surging in the basin to be greater, the velocity of the flow leaving the basin to be higher, and the wave action to be greater than would otherwise have occurred. Also, the velocities throughout the excavated channel were higher because of the steep water surface slope between the end of the basin and the river channel (figure 58B). Had the true tail-water

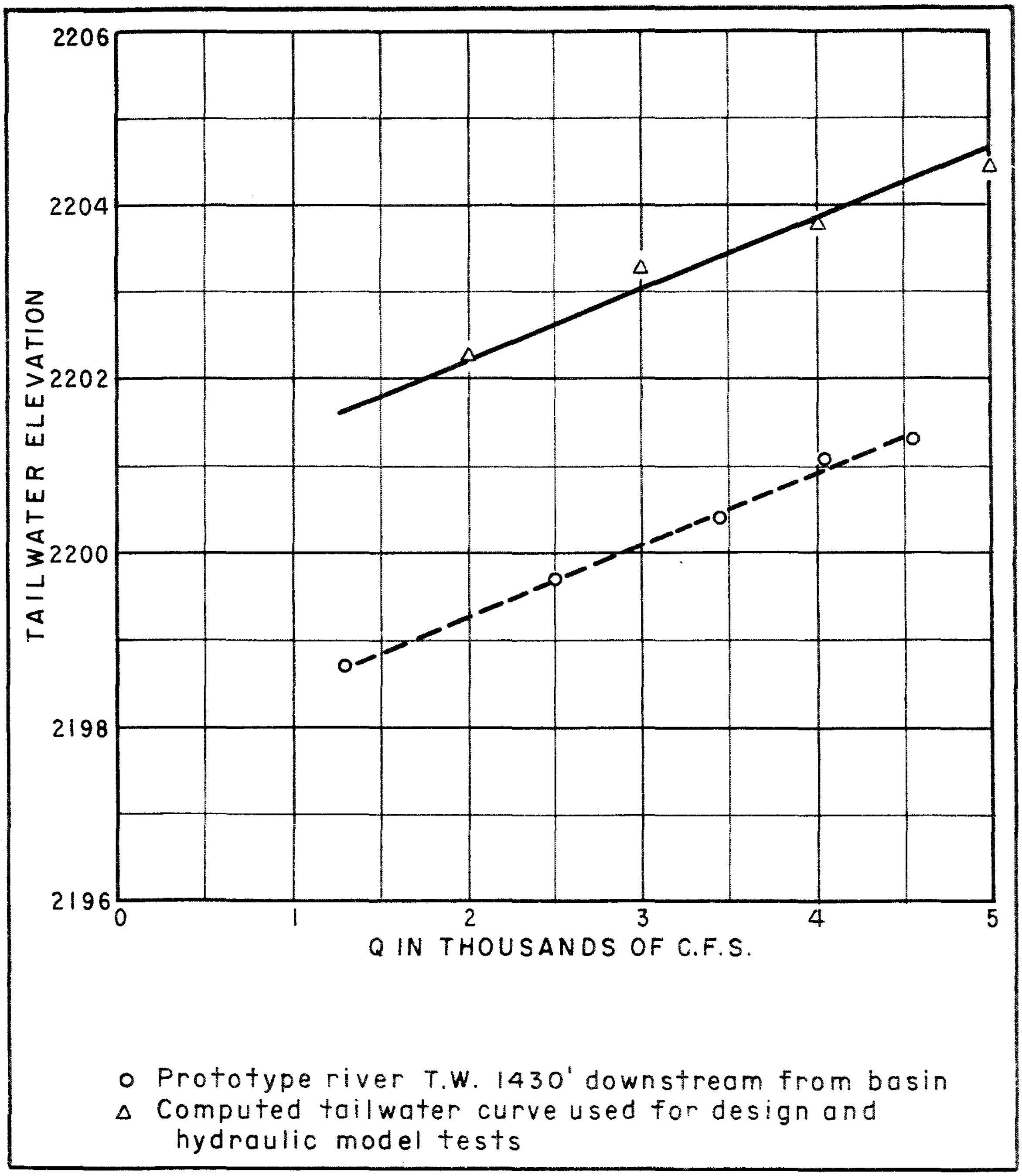


FIGURE 59—Computed and actual tail water curves for service spillway at Shadehill.

elevations been known during the model tests, the entire basin would have been set 3 feet lower to eliminate the excessive difference.

It is fortunate that the excavated channel was not sufficiently wide and deep to allow the tail-water at the stilling basin to approach the elevation of the river surface, see figures 55 and 56. Had this occurred the jump would have been near the sweep out point and the energy dissipating ability of the jump would have been greatly reduced. Serious damage to the structure could thereby have occurred.

10. Erosion below the stilling basin.--Despite the low tail-water conditions there was no significant bottom erosion as a result of the flood. Apparently, the bottom velocities after passing through the jump were directed to the surface by the end sill, producing a boil on the surface rather than erosion on the channel bottom. The channel bottom was protected for 100 feet downstream from the end of the basin, however, by dumped riprap 3 feet thick. The riprap was fairly well graded in size from fist-size pieces up to 24 inches in maximum dimension.

Figure 60 shows the contours of the channel bottom after the flood. Although exact elevations of the bottom before the flood occurred are not known, it is certain that no significant erosion occurred. This was as predicted by the model tests.

The model erosion tests were made using a movable sand bed consisting of well graded sand (0.5 to 2.0 mm mean diameter), with no riprap protection on the banks or channel bottom, and also with a movable bed of stabilized sand. The stabilized sand was composed of a cured mixture of sand and cement having a resistance to erosion equivalent to the estimated resistance of theprototype channel material. Both model tests showed bottom erosion about 1 foot below apron elevation at the apron corners for a discharge of 5,000 cfs second-feet (figure 41B). The tests also indicated that severe bank erosion would occur unless ample riprap protection were provided. Bank erosion occurred in the prototype.

The flow after leaving the boil at the end of the prototype apron accelerated rapidly because of the difference in elevation between the top of the boil and the channel water surface. It was this velocity combined with the waves generated at the end of the stilling basin which produced the erosion of the channel banks.

The right bank of the channel consisted of a low riprapped slope more or less parallel to the basin training walls, as shown in figures 60 and 61. The two trees located about 6 and 8 feet from the bank line in figure 61, were on dry land before the outflow started. The left bank was a higher and steeper slope about 15 feet high, and was riprapped for 100 feet downstream. Near the end of the riprap the left bank started a sweeping curve to the right as shown in figure 61.

During operation the riprap on the right bank was soon submerged and the waves began to erode the soft earth. The trees were undermined and a new channel cut to the right of the trees (figures 58 and 62B). The surface velocity and waves attacked the left bank just downstream from the end of the riprap, causing rapid decay of the soft material for several hundred feet downstream. A sizable area was eroded at the end of the riprap, causing an eddy to form in the area. This action became progressively more violent and the eroded area enlarged until the riprap was attacked from behind. The effectiveness of some of the riprap was lost, since it slumped into a low pile, see figure 62A. It was estimated that the left bank line downstream from the riprap had receded as much as 80 feet as a result of the combined velocity, wave, and eddy action. The waves that were partly responsible for the left bank damage were similar to those shown in figure 58.

The bank damage in itself was considered inconsequential in that the land in the immediate area was expendable. However, the bank damage caused concern because of the possibility of the tail-water being further lowered and further aggravating operation of the stilling basin. The extent of erosion of the banks is evident in figure 60. To repair the bank damage, a band of riprap 50 feet long was placed across the channel bottom and up the channel banks. The right bank, figure 62B, was replaced to an elevation higher than the original and riprapped to prevent widening of the channel. Riprap was also placed along the left bank beyond the proposed 50-foot

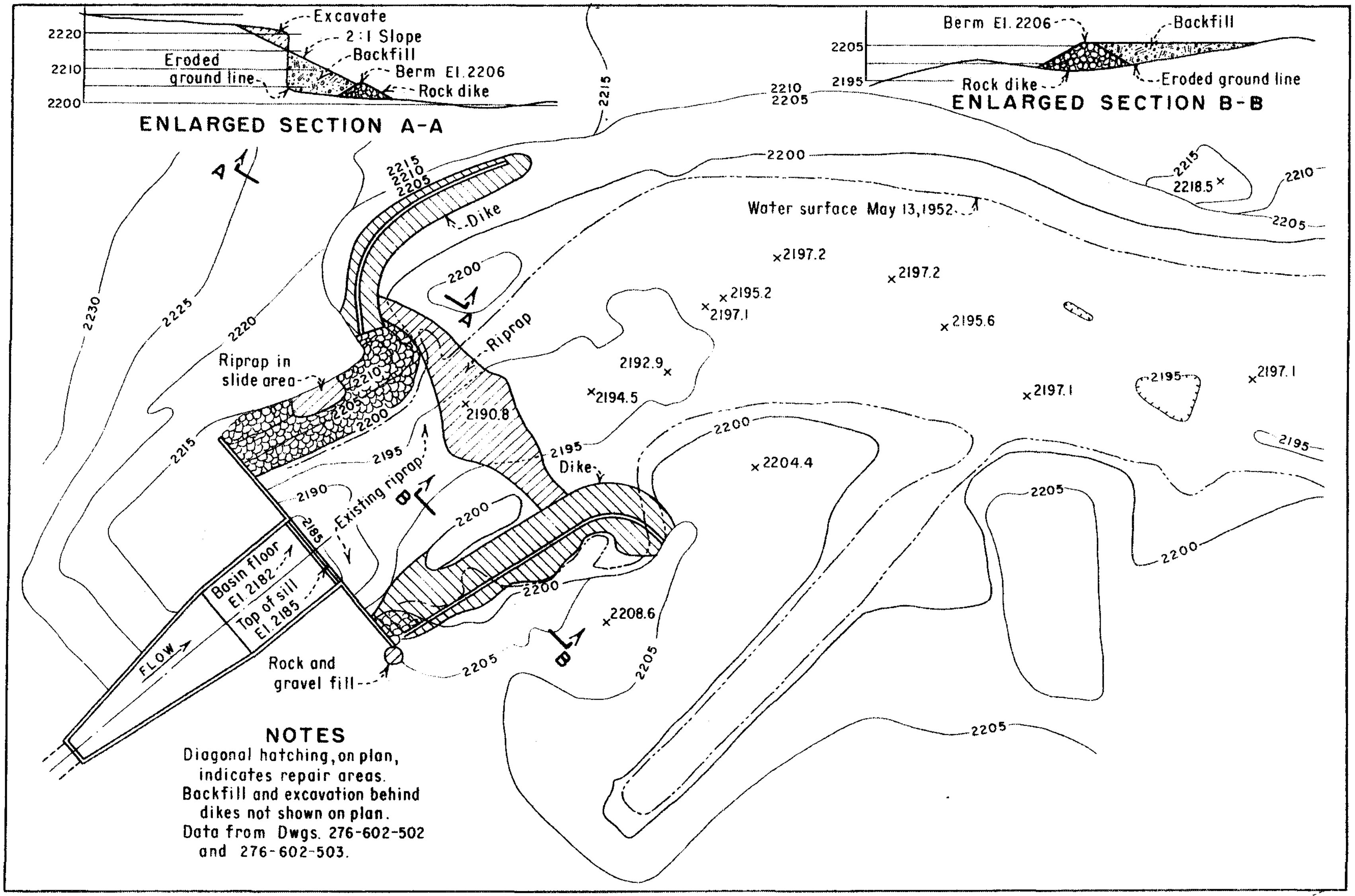
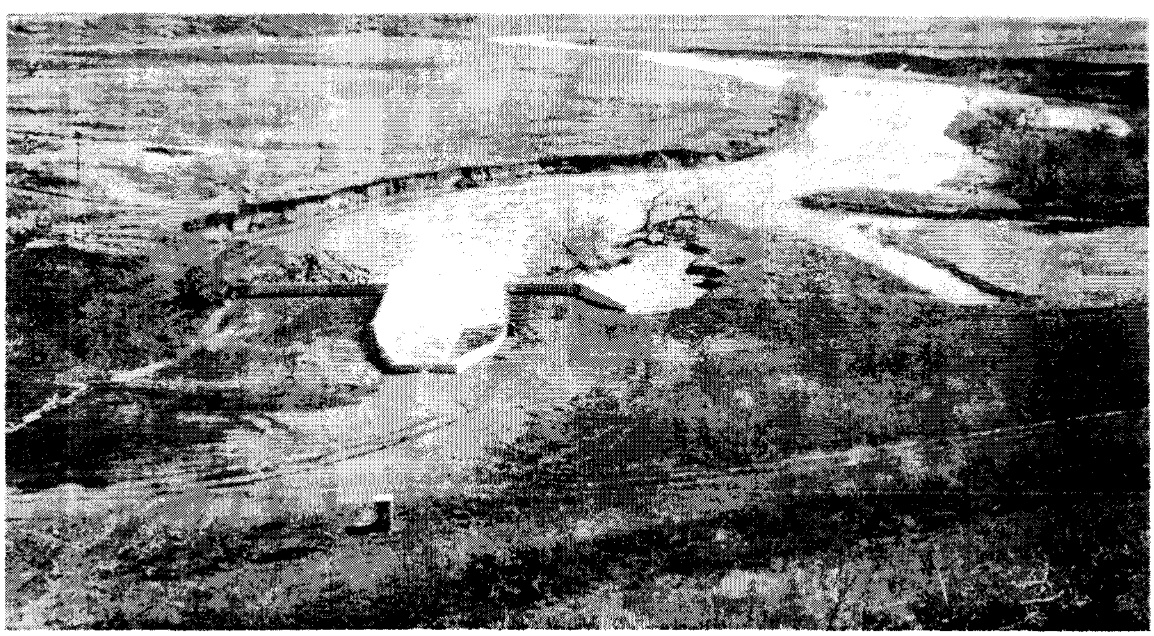


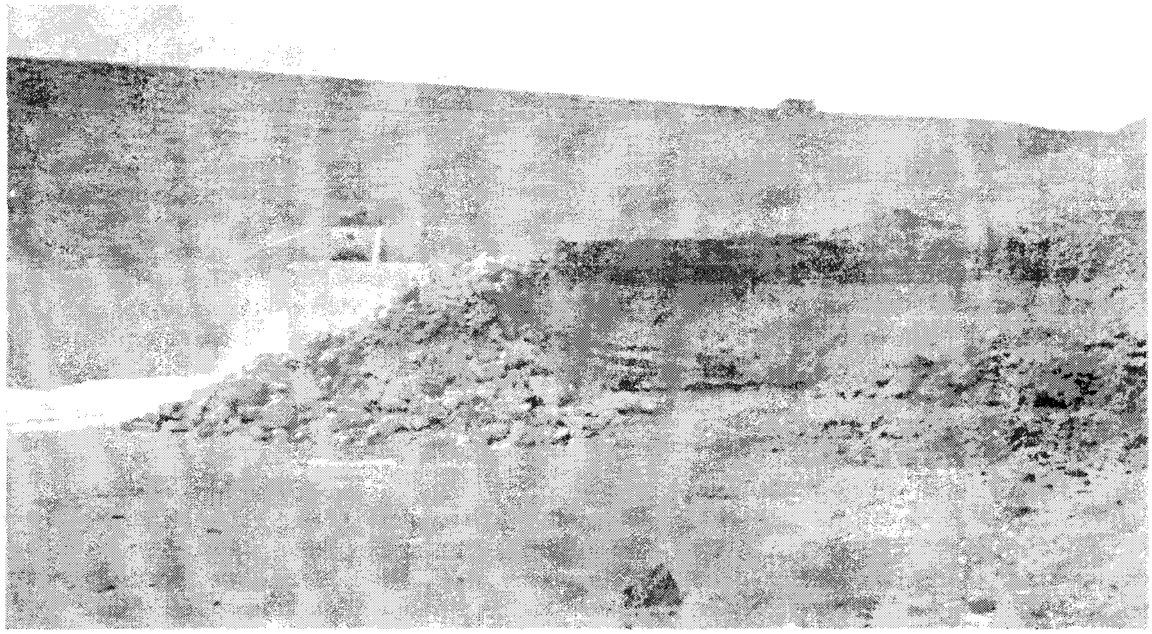
FIGURE 60--Flood erosion and repairs to spillway outlet channel following 1952 flood.

Before Flood
Downstream area before significant bank damage had occurred.

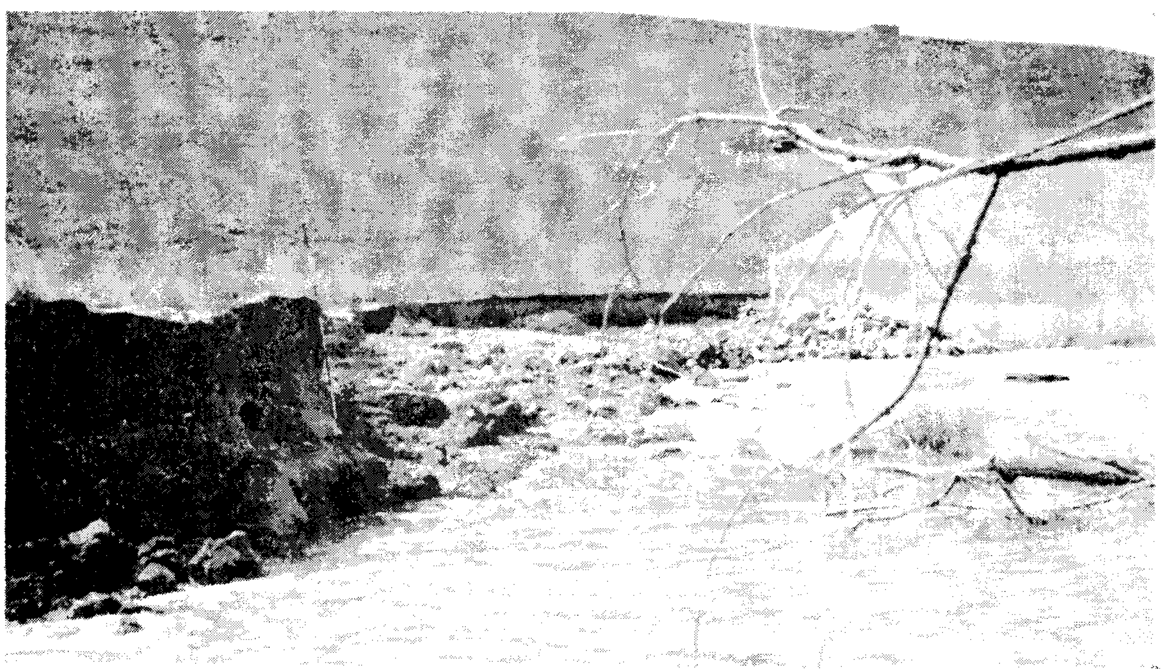


After Flood

FIGURE 61--Channel erosion caused by 1952 flood at Shadehill Dam.



A--An eddy in the foreground area caused erosion of the bank. Riprap was attacked from behind.



B--Low right bank was completely disintegrated. Trees to which the branches at right belong were on dry land before the flood.

FIGURE 62--Erosion of excavated channel banks by 1952 flood at Shadehill Dam.

extension and in other locations where it appeared that the existing riprap needed reinforcing. These repairs should prevent further lowering of the tail-water during future floods.

Inspection of the Prototype After the Flood

After flow through the spillway had ceased and it was possible to enter the spillway tunnel, engineers from the project inspected the exposed surfaces of the concrete structure. Concrete generally was found to be in very good condition and the service spillway should pass other floods of equal magnitude of the 1952 flood with no serious damage. Indications are that the major portion of the rather superficial damage in the tunnel was caused by disintegrated portions of a safety boom of concrete block anchors, cables, and oil drums. Portions of this boom remained dangling in the spillway entrance after the flood while other portions had broken cables and passed through. No evidence of cavitation was found.

ACKNOWLEDGEMENTS

Both the Heart Butte and Shadehill run-offs occurred without warning at isolated sites located about 900 travel miles from the Hydraulic Laboratory. At Heart Butte, general flooding of a large area outside the area protected by the dam caused the road to the dam to be flooded and access to the project was difficult. Prototype test equipment was not available at the dam and trained personnel were not available to make as many observations as might have been considered ideal. At Shadehill, personnel had to be brought in from Huron, South Dakota, a distance of about 300 miles.

The Bureau of Reclamation offices in Bismarck, North Dakota, and Huron, South Dakota, performed excellent jobs of supplying observers to record data throughout the run-off period. Without their wholehearted cooperation, this report would not have been possible. Observers working out of the Glen Ullin Office at Heart Butte Dam and out of the Huron office at Shadehill Dam supplied most of the data and photographs included in this report and some of the motion pictures of the spillways in operation. The Hydraulic Laboratory contributed to the program by outlining the tests to be made, by supplying equipment necessary to make measurements, in working up the data and making the final comparisons. The United States Geological Survey cooperated in the program by supplying men and equipment to make the river gaging measurements.

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